THE SETTLEMENT OF CRUSHED ROCK

A THESIS

Presented to

The Faculty of the Graduate Division

by

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In Partial Fulfillment

of the Requirements for the Degree

Master of Science in Civil Engineering

Georgia Institute of Technology

September, 1963

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Approved:

Date approved by Chairman: 10/1/63

ACKNOWLEDGMENTS

The writer wishes to express his sincere appreciation to Professor George F. Sowers for his guidance and encouragement during the course of this research.

To Dr. Aleksandar Vesić and Dr. Horace W. Straley, III, many thanks are extended for helpful comments throughout the research and during the preparation of the text.

The writer wishes to thank all those whose interest and help aided in this project. Particular thanks are due Professors R. M. Dinnat, B. B. Mazanti, R. M. Scholtes and A. E. Schwartz for their comments and discussion during the course of the research.

Thanks are extended to Mr. Romeo Martin for his preparation of the thin sections of rock.

To my wife, Nell, I express deep gratitude for her patience, encouragement and assistance which made this work possible.

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SUMMARY

It has long been recognized that rock fills settle at a constantly decreasing rate with time; however, there is no general agreement among engineers as to what are the major factors influencing the rate and amount of settlement, and what causes the settlement to be a time-related phenomenon. The purpose of this research was to investigate these two problems by laboratory testing of the settlement of crushed rock, by studying the effect of a constant, sustained load on a point of rock bearing against rock, and by determining if creep and strain-related failure occur in rock which exhibits time-related settlement in the fragmented state.

Consolidation tests were run on well-graded samples containing particles varying from 2.5 to 3.8 centimeters in greatest dimension, to material retained on a Number 8 or 2.38 millimeter sieve. The samples were placed three inches thick in a consolidometer measuring 7.5 inches in diameter. The ends were capped to help distribute the load to the mass effectively. The samples were subjected to incrementally increasing loads up to 32,595 psf, while various programs of flooding and high-pressure jetting were carried out in the tests to investigate the effects of various construction operations. Two materials were used in the tests: Nantahala Graywacke (a dense, metamorphosed, feldspathic sandstone), and Pottsville Sandstone.

The point-crushing tests were performed on graywacke points bearing on graywacke slabs and steel plates, and subjected to 100 pound loads. Both dry and inundated tests were run.

Cylinders of graywacke were subjected to sustained, constant loads in an attempt to produce time-related failure of the cylinders. In addition, attempts were made to measure creep in the rock.

The consolidation samples underwent immediate settlements of between two and four per cent of their original heights, depending on material, void ratio and whether they were initially dry or saturated. In each test, settlement then continued at a gradually decreasing rate, which very nearly followed a straight line in a logarithm of time versus settlement plot. The tests indicated that settlement of crushed rock takes place through fracture of point and edge contacts between particles in the mass, resulting in movement of the particles. The movement of every particle affects the conditions for the equilibrium of the surrounding particles, resulting in a gradual settlement of the mass as the particles assume new and more stable positions.

Comparison of consolidation tests between this research and previous work done indicates that an increase in uniformity and size of rock particles in a rockfill may result in greater immediate settlement and a smaller rate of secondary settlement. It appears that the rate of secondary settlement may be independent of the rock strength. The application of water to a dryly constructed rockfill increases the settlement, and the manner of application does not seem to be significant. Presaturation of the particles was shown to have about the same effect on settlement as the application of water by jetting or flooding.

Mainly due to time limitations, the writer was unable to measure creep satisfactorily; however, there were good indications that creep could be a factor in the settlement of rockfills, as apparent strain-related failures were obtained in the graywacke cylinders, and the points of rock broke repeatedly under the sustained loadings during the point-crushing tests.

A microscopic examination of thin sections and sheared surfaces of the graywacke indicated that the mica content may be a determining factor in the strength of the rock, which seemed to be due more to contact adhesion between the grains than cementation or fusion of grains. It is hypothesized that failure in the graywacke due to unconfined compression of either short or long duration, is caused by movement between the grains until orientations of mica flakes along prospective shear planes occur.

An investigation into the cause of rock weakening by water is imperative to the continuation of this research.

CHAPTER I

INTRODUCTION

Statement of the Problem

An understanding of the settlement characteristics of large masses of crushed rock such as are present in rockfill dams is becoming a subject of increasing importance to the engineering profession as dams are constructed to ever-greater heights. Of particular significance is the long-term settlement that continues long after completion of the dam, because it can cause serious damage to the core-wall or membrane of a dam, thus endangering lives and property when a water load is on the dam. In addition, the competence of a rockfill is judged in part by its settlement record which reflects the construction techniques used as well as the quality of the rock itself.

The settlement records of existing dams and previous research on this subject (1) both indicate that the continuing settlement decreases with increasing time and, in fact, closely resembles the secondary settlement portion of a one-dimensional consolidation curve for soil. Unlike most soils, however, the rockfill settles noticeably for many years. For example, Dix River Dam in Central Kentucky, built of dense, compact, finely crystalline limestone, has settled continously for 37 years since its completion and indications are that some settlement will be noted for the next 10 to 20 years (2).

While long-term settlement has long been recognized and excellent records of settlement have been maintained for most of the major rockfill dams in the world, very little rational analysis of the actual mechanics of the settlement has been made. No one has ever seen the interior of a rockfill and undisturbed sampling is impossible using present methods, due to the nature of the material.

There has been a good deal of discussion concerning the advantages of various techniques used in constructing rockfill, such as placing, dumping and dropping of the rock. The value of sluicing the rock with high-pressure jets of water as it is put into position is almost universally accepted; however, the actual mechanism by which sluicing causes rapid fill consolidation is a subject of much debate and has lead to contradictory statements in the literature.

Since the settlement curves for rockfills closely resemble the secondary portions of one-dimensional consolidation curves for clay, it would appear that both might be manifestations of the same phenomena. Many theories have been advanced to explain the mechanism of secondary settlement of soils, particularly clays, although none have been well substantiated. Among the various causes that have been suggested for secondary effects are the gradual readjustment of frictional forces (3), the jumping of clay bonds (4), viscous structural reorientation caused by shear stresses developed during primary consolidation (5) and the readjustment of grains delayed by viscous resistance in the oriented or "bound" water (6), although the role of the "bound" water has been seriously questioned (7). It appears that none of the above causes would be applicable to an explanation of rockfill settlement; therefore, either it is an unrelated phenomenon or the real explanation for secondary settlement of clays has not been advanced. In any case, the nature of rockfill settlement is a subject for further research.

Review of Literature

The first major contribution to the technology of rockfill dams was a history and accounting of the status of the technology up to 1939, presented by J. D. Galloway and discussed by 25 engineers (8). Very little was published in the succeeding 20 years concerning rockfill dams with the exception of a few isolated papers presenting further results on the performance of existing dams, and some chapters in textbooks which presented mainly summaries of the Galloway paper and discussions. The next major contribution was the Symposium on Rockfill Dams (9) which was compiled by the American Society of Civil Engineers and contains 24 papers with discussions written about various dams around the world. It was published in 1960 and thus presents a fairly recent accounting of the status of rockfill dam technology.

Both of these sources contain valuable information on construction practices and post-construction records, as well as opinions by many eminent engineers as to the mechanics of rockfill settlement. Unfortunately, with the exception of one discussion by Terzaghi (10), the authors never went further into the subject than opinions and deductions based on past observations. The result is a number of discrepancies and some contradictions.

Load Transfer Through the Mass

It is generally agreed upon among most civil engineers that the load due to overlying rock and water is transferred through the mass of rockfill by means of point and edge contacts, which fracture when the stress in them becomes greater that the compressive strength of the rock. Presumably, when the surface area of the now-blunted point or edge becomes great enough to reduce the stress in the rock contact below the critical stress, equilibrium will be reached.

Terzaghi, 1945 (11), has concluded from the results of compression tests on a considerable number of quasi-isotropic, non-metallic materials, including marble and cement mortar, that the failure of these materials at low confining pressures is primarily due to tension and shear between the grains. Since this failure takes place around the grains, it will undoubtably leave an irregular interface that may have no greater contact area than the old surface had. Thus, more than one fracture of a single point or edge may occur until sufficient contact area is obtained to transmit the load.

There is no reference in the literature to the possibility of creep and strain-related failure, specifically in regard to rockfill settlement. Bridgman, 1952 (12), McHenry, 1943 (13) and Nadai 1950 (14) all report, however, the tendency of non-metallic materials to creep or exhibit strain while maintaining load when subjected to a continuing high stress. While the creep or time-dependent strain of the individual rock fragments is undoubtedly only a small fraction of the settlement, it may lead to failure of the highly stressed rock points and edges when a critical amount of strain is reached.

The role of rock fines and dust in the transfer of load through a rockfill mass and the accompanying settlement of the mass is a subject of some argument. Some engineers feel that the presence of finer material in the voids between the larger rocks will add stability to the mass as their additional contact points will help share the load. Muckleston, 1939 (15), states "The cause of settlement and inequality of settlement can both be greatly minimized if the voids in the rock are filled with a fairly well-graded but coarse material such as a mixture of coarse sand and quarry waste, ... which is sluiced into the mass." On the other hand, Galloway, 1939 (16), comments "...any wide divergence in size will cause excessive and unequal settlements.", and Howson, 1939 (17), states, "The larger the rock, the fewer will be the bearing points for any given depth of fill, resulting in less settlement."

The Purpose of Sluicing

It is impossible to discuss the role of the finer rock material in a rockfill without mentioning sluicing--the most controversial of all subjects in rockfill technology.

Sluicing can be defined as the application of water under pressure to the surface of a rockfill during construction. The normal procedure is to apply the water by means of hydraulic monitors in streams of 2 1/2 or 3 inches in diameter, with a minimum nozzle pressure of 75 psi. The jets are directed toward the face of the slope so as to strike the rock as it is being dumped.

The benefit to rockfill settlement derived from sluicing was perhaps emphasized best by an incident that occurred in the winter of 1933. Cogswell Dam, a 280 foot high rockfill dam located on the West Fork of the San Gabriel River in Los Angeles County, California, was approximately 80 per cent complete. It was constructed of placed rockfill that had not been sluiced, due to a scarcity of stream flow. The rock was a granitic gneiss with an average compressive strength of 6600 psi. On December 31, a major storm swept in from the Pacific Ocean and yielded 15.07 inches of rain at the dam by the next day. The dam immediately settled some four per cent of its height, greatly damaging the exterior packed-rock section and the concrete facing. The storm replenished the water supply so that sluicing could now be undertaken, and after several months of pumping of clear water into the rockfill, the settlement had increased from four per cent to six per cent of the height of the dam.

After this incident, the value of sluicing was generally recognized and most specifications for rockfill dam construction now include provisions for sluicing with clean water anywhere from two to four times the volume of the rock, at pressures ranging from 75 psi to 200 psi.

There still is some question as to whether sluicing is necessary when no fines are present. Spielman, 1958 (18), in his discussion of the principal methods for building successful rockfill dams states "The method ...is to put only large pieces of clean sound rock into the fill. Sluicing is not required in this method because large stones bear directly on large stones and little settlement is likely." However, Porter, 1958 (19), notes that, "Even a fill of clean large rock when properly sluiced is given a degree of beneficial lubrication, and settlement during construction is aided."

The concept of lubrication of rock particles by water is frequently mentioned by many different authors as being one of the benefits of sluicing; however, Terzahi, 1958 (20), recalls that " ...it is known that the wetting of unpolished rock surfaces has no effect on the coefficient of sliding friction of rock on rock." This statement is substantiated by experiments undertaken by Tschebotarioff and Welch, 1948 (21).

If water is accepted as being a non-lubricant of rock, then another phenomenon must occur upon the application of water to a rock mass, for the settling effect cannot be denied. Terzaghi is of the opinion that the most vital benefit derived from sluicing is the weakening of desiccated outer portions of the rock fragments, due to saturation, thus causing the mass to settle more initially and less upon completion of construction (22). He reports, 1945 (23), that the compressive strength decreases between 10 and 20 per cent in igneous rocks upon immersion in water; and, "...there are no known rock constituents which decrease in strength upon contact with water." Therefore the loss of strength can only be due to one of the two following conditions:

(1) Part of the intercrystalline matrix consists of soluble material which goes into solution when dry rock is immersed, whereby the intercrystalline connections are weakened.

(2) Part of the strength of the rock is due to the surface tension in that part of the interstitial liquid which does not evaporate at room temperature. Immersion eliminates this source of strength in the same way it eliminates the entire strength of a dry clay.

The additional benefits derived from high-pressure sluicing as opposed to low-pressure applications of water are still being discussed. Bleifuss and Hawke, 1958 (24), who are advocates of sluicing with powerful jets of water, observe:

> When water is properly applied, the effect on a mound of freshly dumped rock is amazing. It seems to dissolve and flatten out, and there is not much rolling of rock down the dump face. Rather there are frequent slides. Segregation of sizes is minimized.

Another widely-held concept of the value of the sluicing process is expressed by Snethlage, Scheidenhelm and Vanderlip, 1958 (25). They believe that the purpose of sluicing is to cause the pieces of rock to come to rest more nearly in the final position and to move finer material away from the points of contact between larger pieces, thus yielding firmer contacts. Therefore washing of fines into the interstices is a result, not the purpose, of sluicing, and will not result in compaction of the fines to an extent where they will resist materially the settlement of the surrounding rockfill (the concept stated above by Muckleston).

Terzaghi, 1958 (26), observed tests on a sample fill in which an attempt was made to plug the voids in a rock mass consisting of stones up to 2 1/2 feet in size by sluicing a mixture of sand and gravel into the voids. The fines immediately clogged the voids near the top layer, preventing any further movement of fines through the mass, while the energy of the jets was dissipated upon striking the top of the rock mass. Thus the interior of the fill received no more benefit from the water jets than it would have from a heavy rainfall. Hellstrom, 1955 (27), supports this concept of low-pressure application of water with his report on a 15-meter-high dam built of placed rockfill. When flushed with water (at no greater pressure than a hard rain) the dam settled 12 per cent.

Terzaghi further observes, in direct contradiction to the observation of Bleifuss and Hawke above:

> Hence, as a result of dumping and sluicing, the grain-size characteristics of the dike-shaped lift change from relatively fine grained and well graded material at the top to coarser and less well graded material near the bottom of the lift. Yet even within the top layer the action of the monitor is far from producing a radical modification of the texture of the fill, comparable to the effects of compaction.

It is obvious that the rockfills observed by Bleifuss and Hawke and the one used in the test that Terzaghi observed differed in properties such as material, strength, amount and size of fines and size of voids. In all probability the pressure of the jets varied in the different cases.

It is unfortunate that no more details were given as such divergence of observed results of the same phenomenon is unusual.

The value of high-pressure jets of water in displacing pockets of clay and other local discontinuities in the mass--when they are exposed to the full force of the jets--cannot be denied and has been reported by many authors.

The Time-Lag of Settlement

Very little attempt has been made by engineers to explain the time-dependent character of rockfill settlement. Most of them simply recognize and accept the fact that rockfills settle continuously over a period of many years, with the greatest amount of settlement taking place during and shortly after construction.

Sowers, 1962 (28), ascribes the settlement of rockfill to two phenomena:

> First, the migration or working of fines from between the points of contact between the larger rock permits the rocks to re-orient themselves and assume a more dense structure. Second, the crushing of the contact points between the larger rocks under the extreme stresses developed by the embankment weight causes the rocks to move and develop new points of contact which in turn crush again.

The work of Williams, 1963 (1), would tend to eliminate the first phenomenon above as being a cause of time-dependent settlement, since the settlement characteristics of rockfill were reproduced in the laboratory using clean rock of fairly uniform size.

Terzaghi, 1958 (29), first noted this time-lag in 1920 in connection with an experimental investigation of the compressibility of cohesionless sand. His analysis of the process is as follows:

> ... the sand layer in its initial state contains many grains which come to rest, during the process of pouring the sand, in a rather unstable position. When a load is applied they rotate into more stable positions. The movement of every grain effects the conditions

for the equilibrium of all the others, and such chain reactions inevitably require a considerable amount of time. Since the number of particles in relatively unstable positions decreases with time, the rate of settlement due to readjustments also decreases with time. The progressive settlement of rockfills is caused by similar processes.

It would appear that the crushing of point contacts, rather than the rotation of particles (or possibly both) might be a better explanation for the movement of the individual particles in the case of rockfill.

Baumann, 1958 (30), concludes "...residual or post-construction settlement could only take place through additional compression of the crushed material in the voids once point contact has been eliminated." This conclusion, based on a mathematical analysis of rock volume in the hauling trucks during construction of Cogswell Dam and the in-place volume of the dam, does not seem logical. It is difficult to conceive of one rock being lifted off its contact point with another rock by their own debris. Surely, if this could occur, settlement of the crushed material would restore the point contact again.

A time-dependent fracture of the intergranular matrix of the rock, due to critical strain being reached as mentioned above, and the resulting movement of the particle would be consistent with the explanations of both Sowers and Terzaghi.

Investigation of the Rock Properties

Sowers (28), pp. 250-252, suggests that tests be made on the proposed rock to determine durability and quarrying characteristics. These tests should include wet-dry and freeze-thaw (where applicable) compression tests, tests for friability and shear strength and petrographic analyses. Also examinations of old structures constructed of the same material might prove of value in evaluating durability.

A review of the post-construction reports on many dams indicates that rarely have such complete investigations been made of the materials used in the dams. Of the 35 or so dams discussed in the Symposium on Rockfill Dams, the engineering properties of the rock are included in only two cases, (31) (32), and a comparison of the wet and dry compressive strength of the rock is mentioned in just one of these.

Purpose of the Research

In this study the author proposes to:

(1) further refine the process of duplicating the settlement characteristics of large rockfills using laboratory model consolidation tests,

(2) determine the difference in settlement characteristics between samples of uniform and well-graded crushed rock,

(3) study the difference in settlement characteristics of dry and pre-saturated crushed rock,

(4) study the effects of high-pressure sluicing and flooding of dry and pre-saturated crushed rock,

(5) observe the performance of an isolated point of rock subjected to a constant load, and

(6) investigate the possibility of creep and strain-related failure being a factor in the settlement of crushed rock.

The results of the research will be largely qualitative rather than quantitative due to the nature of the tests and the inherent difficulties involved in setting up tests. For example, it will be impossible to form consistently uniform samples of crushed rock so that various consolidation tests may be compared quantitatively. Also the tests will be performed on two rocks whose engineering properties vary from those of any other material tested and even from samples of the same material taken from different locations.

It is hoped that this research will succeed in clarifying to some degree the mechanics of rockfill settlement and the factors influencing it.

CHAPTER II

INSTRUMENTATION AND EQUIPMENT

The Consolidometer

The consolidometer used in the series of consolidation tests was constructed from seamless steel tubing of 7.5 inches inside diameter and 0.5 inch wall thickness. It is 5 5/8 inches high and threaded to a base machined from 1.5 inch steel plate. The loading head was also constructed from 1.5 inch steel plate and designed to fit loosely in the consolidometer. The consolidometer, base plate and loading head were all cadmium plated. For jetting tests, 12 water jets were placed in the consolidometer wall. The top level consisted of six jets located 1.5 inches from the top of the wall and spaced at equal distances around the perimeter. The lower level consisted of six equally-spaced jets at three inches below the top of the wall and offset 30 degrees from the top level. These jets consisted of 1/16 inch diameter holes opening to the inside of the consolidometer and connected by tube fittings to 1/4 inch high pressure nylon tubing leading from the bottom of a stainless steel pressure tank of one cubic foot capacity. Nitrogen at 300 psi pressure was admitted to the top of the tank to provide the necessary pressure to the water. The tank was equipped with a manometer so that the water level could be observed at any time, and the apparatus included sufficient valves to allow complete control over the flow of both nitrogen and water.

The consolidometer was also provided with three equally-spaced 5/8 inch diameter drains located in the wall at the bottom of the sample.

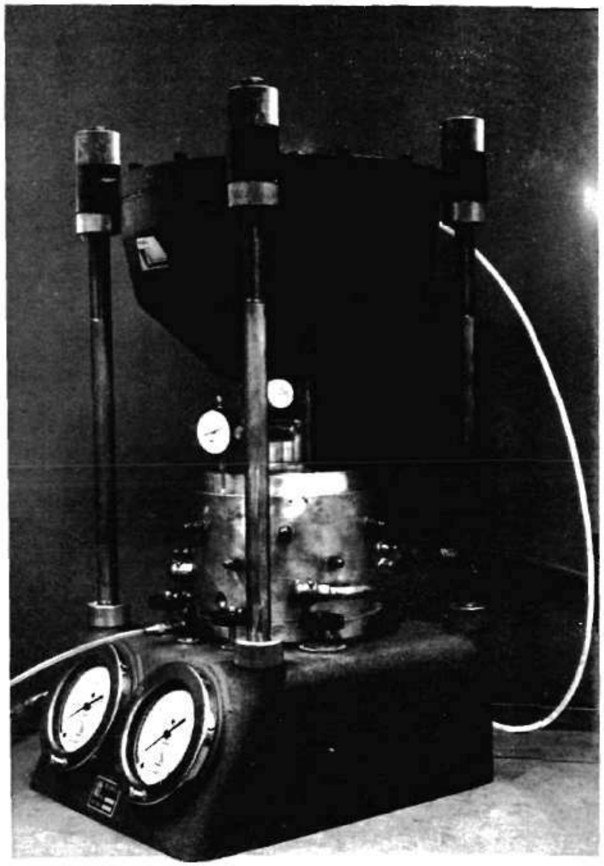

Figure 1. Consolidometer in
Conbel Loading Device

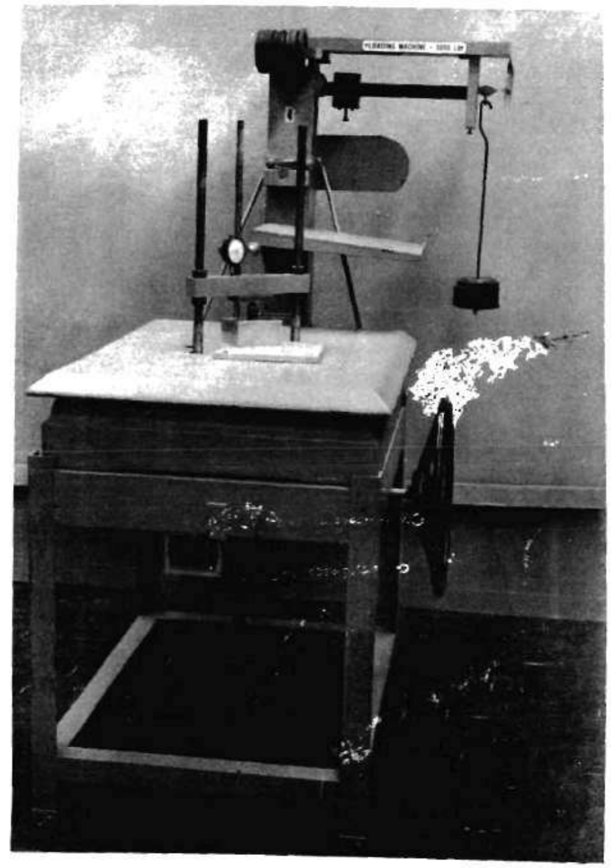

Figure 2. Platform Loading Device

The drains were interconnected by 1/2 inch copper tubing leading through a valve into a rubber hose, which drained the water into a stilling basin. A drain was located in the middle of the base plate of the consolidometer to facilitate complete drainage. It was also used to fill the consolidometer with water during the flooding tests. The consolidometer is shown with the jetting apparatus attached in Figure 3.

Load Application

A constant load--regardless of sample deflection--was applied to the consolidometer by means of a Conbel Loading Device as shown in Figure 1. This device would supply a constant load of up to 10,000 pounds by means of a piston-diaphragm system, when provided with a constant supply of dry air at a minimum pressure of 125 psi. Precision air regulators operated by control valves allowed the use of either a low-pressure loading system (zero to 3000 pounds) or a high-pressure system (zero to 10,000 pounds). Pressure gages with a repeatability of between 1/4 to 1/2 of one per cent indicated the desired pressure which was read from a calibration chart.

Additional Loading Devices

Two controlled-strain loading machines were used in determining the unconfined compressive strengths of the rock and the capping materials. The machines were similar except that one had a capacity of 20,000 pounds and was used in testing the weaker materials while the other had a capacity of 60,000 pounds and was used in testing stronger rock. The 20,000 pound capacity machine was also used for long-term loading of rock cylinders in the creep tests.

Four platform-scale compression devices were used in the point-crushing tests. These devices were capable of applying constant loads of up to 3000 pounds to samples when weights were applied to their balance arms. One pound of weight placed on the balance arm resulted in 100 pounds of compressive force being applied to the loading head when it was brought into contact with the sample a sufficient amount to rebalance the apparatus. This was accomplished with a mechanical screw jack (see Figure 2).

Deformation Measurement

A micrometer dial indicator having divisions of 0.001 inches was used to record settlement of the crushed rock samples in the consolidation tests. It was attached to the ram of the loading device as shown in Figure 3, thus eliminating any error due to uneven settlement and resulting tilting of the loading head.

Micrometer dial gages having divisions of 0.0001 were used to measure deformations in the unconfined compression tests and movement due to fracture and creep in the point-crushing tests.

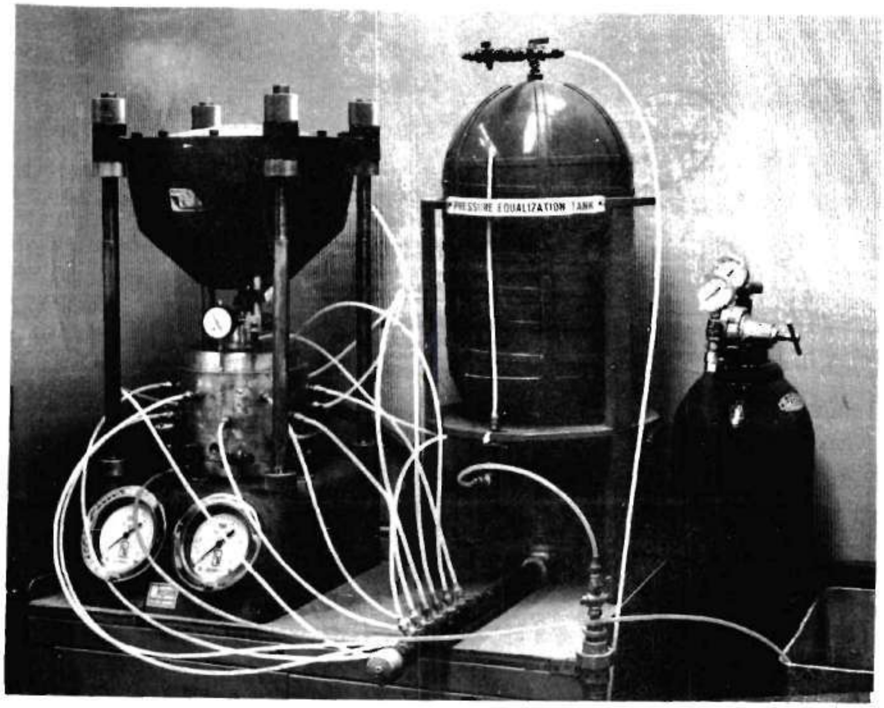

Figure 3. Consolidometer with Jetting Apparatus Attached

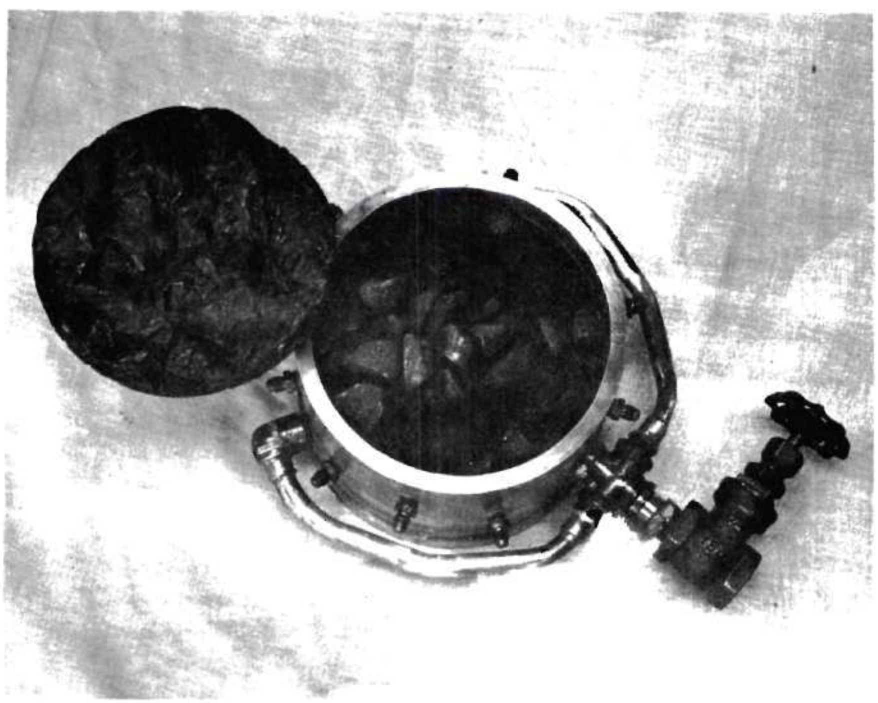

Figure 4. Top View of Consolidometer Showing Crushed Graywacke and Form-Fitted Sulfur Compound Cap

CHAPTER III

TESTING PROGRAM

The testing program can be divided into the following general categories:

(1) the consolidation tests,

(2) the determination of the physical and engineering properties of the rock, including a thin-section analysis of one type of rock used,

(3) point crushing tests,

(4) creep tests, and

(5) control tests on capping materials.

General Description of the Rock

Two types of rock were used in the tests to reduce the variables to a minimum so that the immediate problem of settlement mechanism could be more fully investigated.

The principal rock tested was a metamorphosed feldspathic sandstone called graywacke which was obtained from Nantahala Dam in Southwestern North Carolina. This particular rock was chosen so that the results of this research might be correlated with previous work done on this subject using this material (1). As shown in Table 1, the graywacke was found to be an exceedingly strong material of excellent quality for use in dams.

To investigate the relationship between the settlement characteristics of crushed rock samples of a strong, and a weaker and more porous rock, consolidation tests were also performed using Pottsville sandstone, which is described by Schwartz (33), pp. 25, 26:

> The Pottsville sandstone was obtained from a deep quarry and appears uniform in color and unweathered. Chemically, the stone is nearly pure silica. A cross-section shows that the rock consists of quartz crystals in contact with each other and with small amounts of silica cement present at the points of contact. The quartz crystals appear uniformly clear and colorless, however the entire sample has a buff color due to the presence of a small amount of limonite (1-3 percent).

Table 1. Engineering Properties of the Rocks

Type Rock	Bulk Sp. G.	Density pcf	Average Mod. of Elast. E - ksi	Average Unconfined Strength-ksi Dry	Wet
Nantahala Graywacke	2.78	173	3330	31	26*
Pottsville Sandstone	2.28	142	1710	9	9

Consolidation Tests

Sample Preparation

The consolidation samples were prepared from pieces of rock weighing from 10 to 50 pounds by means of a sledge hammer and chisel. As a result, all pieces were freshly broken on most of the faces. Each sample was composed of a well-graded mixture of sizes, varying from pieces with a maximum dimension of 2.5 to 3.8 centimeters down to that retained on a number 8 or 2.38 mm sieve. Considerable effort was made to obtain pieces having roughly equal dimensions, but this proved to be difficult to control as the pieces became smaller. This was particularly the case with the graywacke where the smallest particles broke along mica planes, giving thin, elongated pieces.

The work of Williams (1) has indicated the definite advantage of using "form-fitted" loading caps on the top and bottom of a consolidation sample of crushed rock to eliminate the local crushing due to flat loading plates. The caps transfer the load to the sample so as to effectively reproduce the action on the interior of an actual rockfill.

For the first test, epoxy resin caps were formed, following generally the procedure of Williams using Shell Epon Resin 828 activated with an adducted aliphatic amine and containing 50 per cent by weight of fine sand to add bulk. A layer of 0.002 inch thick teflon film was placed on the consolidometer base and walls to protect them from the epoxy, which was placed in a 1/2 inch layer. Before the epoxy solidified, a layer of well-graded rock was hand placed so as to become imbedded in the cap. The rock in the cap was not considered to be part of the sample since it was affixed to the cap and could not be included in the post-test grading analysis.

Immediately after the cap had solidified, the sample was placed in the consolidometer and hand-raked to insure good seating of the pieces. Every effort was made to obtain a uniform distribution of sizes with no concentrations of any particular size of rock.

To facilitate the forming of the top cap, a double thickness of a thin plastic film (Saran Wrap) was placed over the top of the sample and fitted to the contours of the rock particles. The epoxy mixture was placed on this plastic film to a depth such that a smooth bearing surface was formed (approximately 1/2 inch). Considerable difficulty was experienced in the forming of this top cap due to the heat produced by the chemical reaction in the large mass of epoxy, causing it to bubble and boil up. This difficulty was not encountered in forming the bottom cap as the steel base dissipated the heat. The problem was finally solved by forming the top cap while the consolidometer was in a freezer maintained at a temperature between 40°F. and 50°F. In addition, a weighted aluminum plate was constructed to fit over the consolidometer and press on the top of the cap. This helped dissipate the heat and prevented the heaving up of the cap during the chemical reaction. The setting time for the epoxy resin to gain full strength was three to five days.

The difficulty encountered in forming and the excessive curing time of the epoxy promped a search for another capping material. A sulfur compound containing a mineral filler and commonly used for the capping of concrete cylinders proved to be satisfactory and was employed for Tests Number Two through Seven.

The bottom cap of epoxy containing the imbedded rock particles was used throughout the series of tests, while a new top cap was formed for each test using the sulfur compound. The capping material was liquid only when heated above about 180° Fahrenheit--very close to the melting point of the previously used plastic film. For this reason aluminum foil was used to cover the rock and prevent the sulfur compound from filling the voids. The advantage of the sulfur compound over the epoxy resin, in addition to its favorable deformation characteristics (Figure 17), was its rapid curing time. The sample was ready for testing within one-half hour after pouring of the top cap.

Tests Number Six and Seven were performed on pre-saturated samples that were prepared according to a method developed by Schwartz (33), pp. 28, 29. The sample, before capping, was placed in a plastic chamber and subjected to a vacuum for approximately 24 hours. At this time (while the vacuum was maintained) distilled, deaired water was injected into the chamber, completely filling it, after which atmospheric pressure was restored. The samples were soaked for at least 48 hours before being placed in the consolidometer and capped. Schwartz reports obtaining saturations of 93 to 98 per cent with Pottsville Sandstone using this method.

Test Number Eight, the control test, was performed without a sample on two 1/2 inch thick end caps--one of epoxy and one of sulfur compound--that were formed in the consolidometer. The epoxy cap was poured in its usual position and the sulfur compound cap was poured directly on top of it. The usual loading head and base plate were used.

Test Procedure

For each test, the consolidometer was placed in the Conbel Loading Device and a seating load of 100 pounds was applied and maintained while the micrometer dial gage was zeroed. Loads of 500 pounds (1630 psf), 2000 pounds (6519 psf), 5000 pounds (16,298 psf) and 10,000 pounds (32,595 psf) were applied in turn and each was maintained until sufficient deflection versus time readings could be made to define the settlement curve.

Flooding of the consolidometer was accomplished by sealing the jet holes in the wall and introducing water into the cell from the bottom drain until the sample was inundated. The flooded condition was maintained until no further effect was noted, at which time the cell was drained through the bottom drain. The water was drained through a collecting basin to catch any fines that may have been washed out of the sample.

Jetting of the sample was undertaken with the drains open so that there was a continuous flow of water out of the cell. When the pressure tank was emptied of water, the process was halted, the tank refilled and jetting continued. This procedure was repeated until three tanks of water had been used, giving a water to rock ratio by volume of 37:1. The drain water was carried through a settling basin before being wasted.

At the conclusion of each test, the sample was subjected to a sieve analysis to determine the change in gradation that took place during consolidation. Care was taken to minimize the mechanical breakdown of the particles occurring during the sieve analysis, particularly with the samples of Pottsville sandstone which broke easily under any kind of grinding action.

An outline for the procedure of each test is shown in Table 2.

Determination of Engineering Properties

Sample Preparation

Various tests were run on the rock and capping materials to determine their strength and deformation characteristics.

Epoxy and sulfur compound cylinders of 7/8 inches up to 2 inches in diameter were cast, using metal tubes lined with teflon film as molds. The ends were ground flat on a 300 grit grinding wheel adapted to a drill press so that they were square with the axes of the cylinders.

Graywacke and sandstone cylinders were obtained using conventional machine tools that had been modified for rock cutting. A twelve-inch diamond tooth circular saw was used to cut parallel faces in the rock, approximately 2 1/4 inches apart. These slabs were then cored with a diamond-impregnated coring bit with an inside diameter of 7/8 inch and an outside diameter of one inch. The coring bit was attached to a water head

Table 2. Outline of Consolidation Test Procedure

Test Number	Rock	Initial Condition	Procedure
1	graywacke	dry	normal loading sequence (500, 2000, 5000 and 10,000 lb)
2	"	"	normal loading sequence--at end of 5000 lb loading, flood
3	"	"	place nominal load (100 lb), flood, drain and continue test
4	"	"	place nominal load, jet and continue test
5	"	"	normal loading sequence--at end of 5000 lb loading, jet
6	sandstone	"	normal loading sequence--at end of 5000 lb loading, flood
7	"	pre-saturated	normal loading sequence--at end of 5000 lb loading, jet
8	----	dry	normal loading sequence

and driven by a drill press. The ends of the rock cylinders were squared by chucking the cylinders in a turning lathe and facing off the ends with a diamond-tipped tool.

Test Procedure

Three epoxy cylinders and three sulfur compound cylinders were measured and then tested in unconfined compression in a controlled-strain testing machine at a strain rate of 0.01 inches per minute. The data were processed by an electronic computer, stress-strain curves were plotted and the average moduli of elasticity of the two materials were obtained (see appendix, Figure 17).

Eight graywacke cylinders were tested dry in unconfined compression to failure at a strain rate of 0.025 inches per minute in a 60,000 pound controlled-strain testing machine. Deflections were measured with a micrometer dial gage reading directly to 0.0001 of an inch. The data were processed on an electronic computer and stress-strain curves were plotted. The average unconfined compressive strength and modulus of elasticity were calculated for the material. An attempt was made to saturate cylinders of graywacke so that a comparison could be made of the dry and saturated strengths, but it was found to be impossible to saturate the rock using the method previously described. Five graywacke cylinders that had been subjected to this process were tested, however, and the results are included in the appendix along with the results of the dry tests.

Similar compression tests were performed on two dry and two saturated cylinders of Pottsville Sandstone to confirm the results of similar tests performed by Schwartz (33), p. 41.

Point-Crushing Tests

Sample Preparation

Four of the previously prepared 7/8 inch diameter cylinders of graywacke were used in this series of tests after the following modification.

Each cylinder was placed in a jig that allowed it to be cut through on a 30 degree angle with the axis of the cylinder, using the twelve inch diameter diamond-tooth circular saw. The cylinder was cut, rotated and cut--the process being repeated until a pyramid-shaped point having five sides was attained on one end. The pointed cylinders were placed in 7/8 inch diameter square-bottomed holes drilled in three inch by four inch by two inch deep aluminum blocks. Each cylinder was embedded in its block one inch and bore against the aluminum at the bottom of the hole. Two of the pointed cylinders and one of the slabs were subjected to the saturation process mentioned above.

A steel cone having a sharp point was also imbedded in an aluminum block in a similar manner to the pointed rock cylinders.

Three slabs of graywacke of about 10 square inches surface area were cut with parallel faces approximately 1/4 inch apart.

Test Procedure

The aluminum blocks holding the steel and rock points were placed in the platform-scale loading devices and held firmly in place by bolts as shown in Figure 5. The following tests were set up: (1) dry rock point bearing against a rock slab, (2) dry rock point bearing against a steel plate, (3) inundated (not saturated) rock point bearing against a rock slab, (4) inundated rock point bearing against a steel plate, (5) steel cone bearing against a rock slab.

For the inundated tests, one inch high pieces of plastic tubing of 1 3/8 inches in inside diameter were affixed to the rock slab and steel plate with petrosene wax to provide watertight "dams" surrounding the points. These containers were maintained full of water during the tests so that the points were constantly inundated (see Figure 6).

A seating load of one pound was applied to each point and maintained while a micrometer dail gage reading directly to 0.0001 was zeroed so as to measure deflection of the point and/or slab. A load of 100 pounds was applied to the point and the immediate deflection noted. Constant surveillance was necessary at first, as whenever there was deflection, a correction had to be applied to the loading device to restore the 100 pound load, Deflection versus time was recorded for each test, which was continued for approximately one week.

Creep Tests

Test Procedure

Two 7/8 inch diameter graywacke cylinders were tested in unconfined compression under sustained loads applied by a 20,000 pound capacity controlled-strain testing machine.

The first cylinder was placed in the machine and a 12,000 pound load (20,000 psi) was applied. After three days, the load was increased to 14,000 pounds (23,333 psi) and maintained for two days, at which time it was increased to 16,000 pounds (26,667 psi). This procedure was continued until a total load of 19,000 pounds (31,667 psi) was placed on the cylinder and maintained until failure occurred.

The second cylinder was tested in the same manner except that the loading was started at 16,000 pounds, maintained approximately one week,

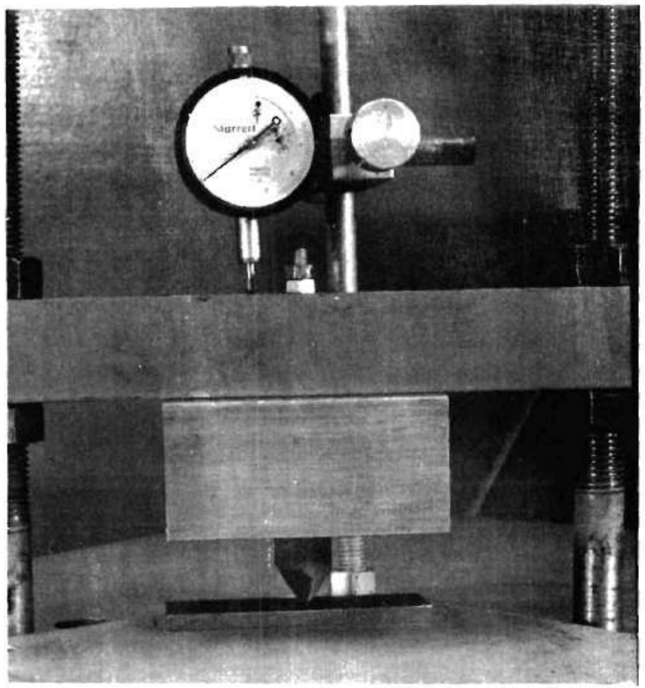

Figure 5. Dry Graywacke Point Bearing on Steel Plate

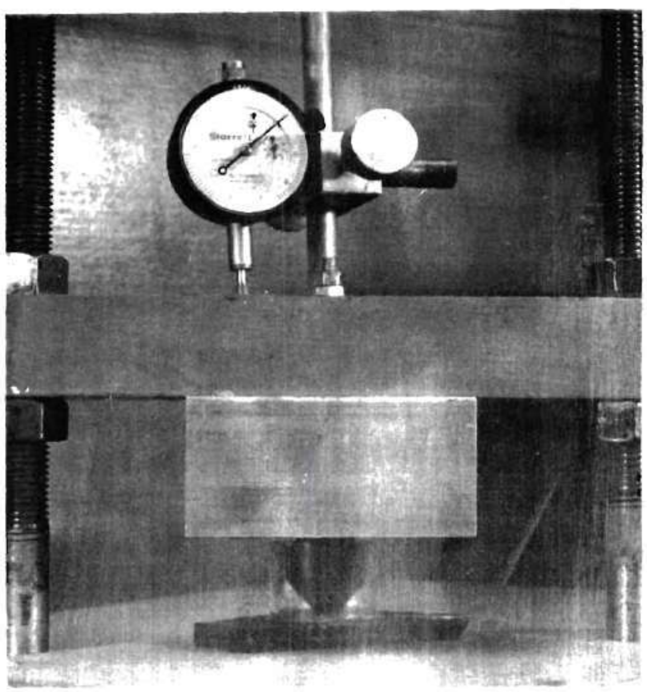

Figure 6. Inundated Graywacke Point Bearing on Graywacke Slab

and then increased to 17,000 pounds (28,334 psi) and maintained until failure.

An attempt was made in the first test to measure deformation in the sample under a constant load using a micrometer dial gage reading directly to 0.0001 inches.

A third test was attempted in which two foil-type SR-4 strain gages (with a gage length of one inch) were affixed to a cylinder of graywacke, using Eastman 910 Adhesive. They were placed equi-distant from the ends of the cylinder on its vertical axis and 180 degrees apart. The strain gages were connected in series so as to indicate twice the strain in the cylinder (one half the indicated strain being the mean strain in the cylinder) and were connected in parallel with two similar compensating strain gages (series-connected) affixed to a slab of graywacke. The compensating gages automatically corrected for temperature variations since the active and compensating gages were all placed on the same type of material. An SR-4 Type J battery-operated strain indicator attached to the gages allowed a direct reading of strain in the cylinder.

Thin Section Analyses

Three thin sections of the graywacke were made from representative samples of the rock that were chosen to include the widest variation in appearance. These thin sections were examined under a petrographic or polarizing microscope to determine the orientation of the grains, the approximate size and shape of the grains and the differences between the samples chosen.

CHAPTER IV

DISCUSSION OF RESULTS

Consolidation Tests

The results of the consolidation tests were plotted, where feasible, as curves having abscissas of logarithm of time, rather than time. This procedure was followed so that the settlement characteristics of the sample during the first few minutes of testing would not be obscured by the small scale that would be required in an arithmetic plot of time. In addition, it was found that the secondary settlement portions of the settlement curves approximate a straight line in a logarithm of time versus settlement plot, and the slopes of these lines can be computed (34).

Where jetting or flooding was undertaken during a consolidation test, the results were plotted as curves having time as the abscissa, because the settlement characteristics of the samples during the period immediately following application of the water would be distorted in a logarithm of time versus settlement plot.

Effect of Gradation

In all the tests, upon application of a new load increment, a crackling sound was heard that lasted just during the immediate settlement--a matter of about a second or two. These were not the "loud snapping sounds" that Williams (1) reported, which continued for several minutes after loading. Apparently the difference in this initial settlement between the present tests and those of Williams can be attributed to

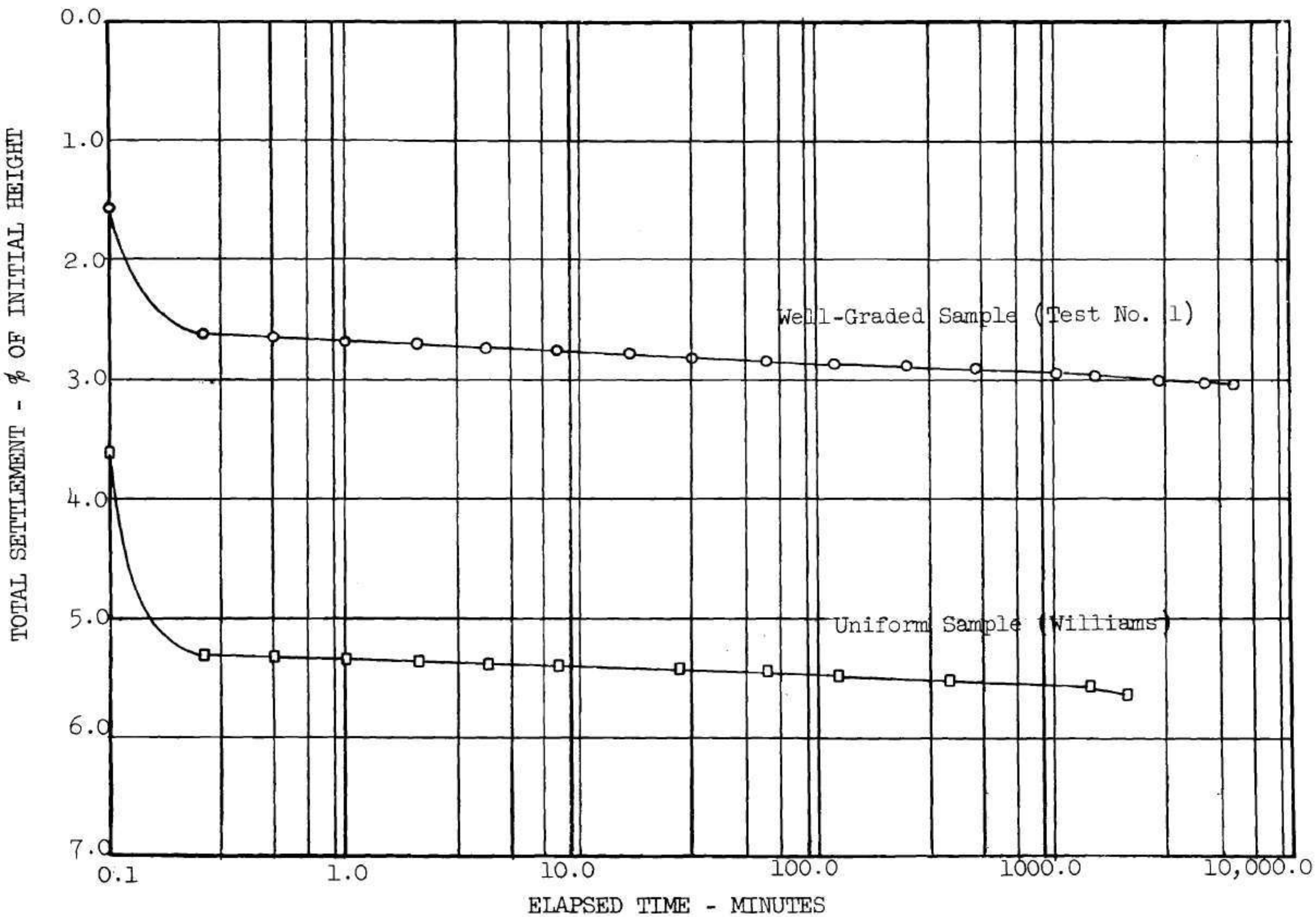

Figure 7. Log Time - Settlement Curves for 16,298 psf Loading - Showing Gradation Effect

the difference in gradation of the consolidation samples, which is readily apparent in the initial void ratios of the two samples. The initial void ratio of the mass in the present Test 1 was 0.63 whereas that of William's Test 3 was approximately 1.66. This great difference is reflected in the log time-settlement curves of Figure 7 which indicate that the initial or immediate settlement of the uniform sample of graywacke is almost twice that of the well-graded sample. This is a logical result, since the well-graded sample has many more points of contact to share the load and the settling effect on the mass of the fracture of one point is considerably less than it would be in the uniform sample having fewer bearing contacts.

The larger immediate settlement of the uniform sample, however, suggests that its rate of secondary settlement may be less than the rate of secondary settlement of the well-graded sample. The number of rock contacts fracturing and disturbing the equilibrium of surrounding particles is less in the uniform sample, thus the mass attains equilibrium sooner than would a mass comprised of many more contact points of widely varying size--all fracturing at different times. This is also substantiated by the curves of Figure 7 which show a slope of -0.096 for the secondary settlement portion of the well-graded sample curve as compared to a slope of -0.062 for the corresponding portion of the uniform sample curve.

It would be unwise to draw any conclusions from the results of such limited testing, but indications are that an increase in uniformity and size of rock particles results in greater immediate settlement and a smaller rate of secondary settlement--a phenomenon of considerable importance in rockfill dams, if true.

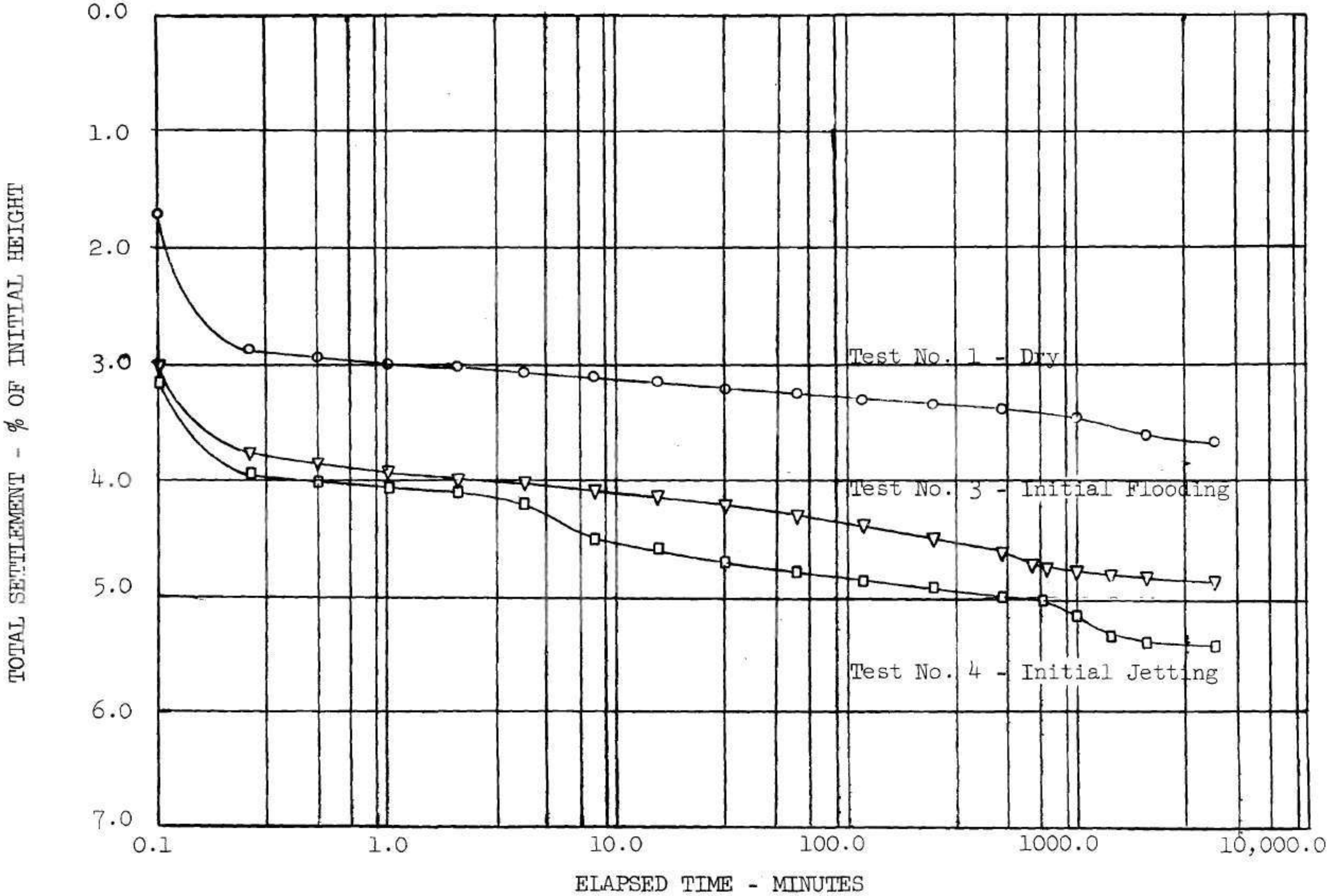

Figure 8. Log Time - Settlement Curves for 32,595 psf Loading - Nantahala Graywacke

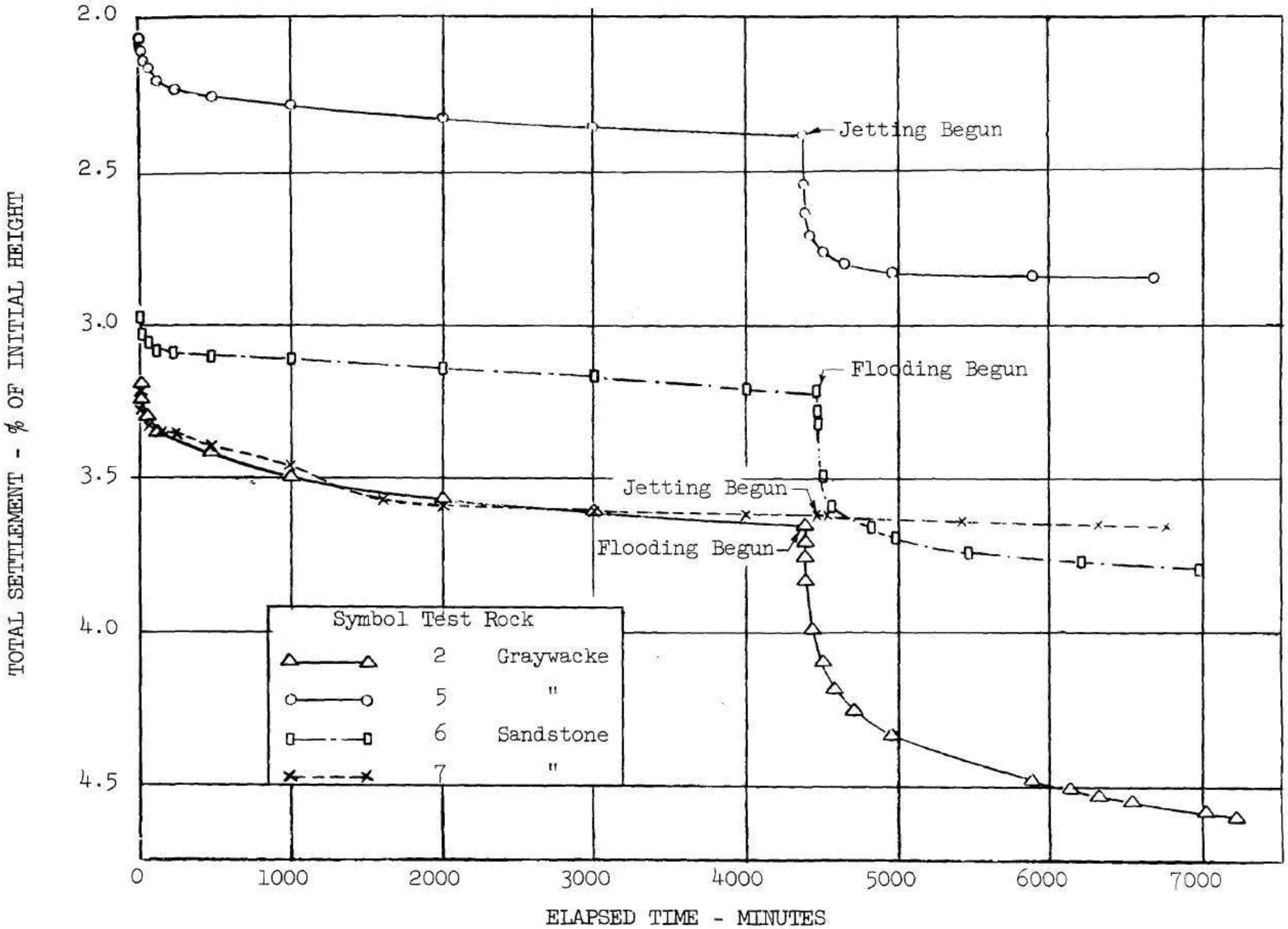

Figure 9. Time - Settlement Curves for 16,298 psf Loading - Pottsville Sandstone and Nantahala Graywacke.

Effect of Jetting and Flooding

As shown in Figure 8, the initial application of water to the graywacke sample by flooding resulted in an increase in settlement of 33 per cent over the settlement experienced by the dry sample; while the initial application of water by jetting resulted in an increase in settlement of 48 per cent over that of the dry sample. Thus the jetting appeared to have some additional settling effect over the flooding. The stepped appearance of the jetting time-settlement curve indicates local settlement or the breakdown of arches in the mass. Since this effect was not evident in the flooded test to the same degree, it would appear that the jetting moved some of the fine material so as to cause unstable arrangements within the mass that broke down upon application of a continuing load.

The results of flooding and jetting of dry samples after considerable secondary settlement had taken place under the applied load are shown in Figure 9. The effect in each case on the settlement was as if the applied load had been greatly increased. Immediate settlement took place at a constantly decreasing rate, gradually approaching the same rate of settlement as existed before the application of water. The increase in settlement of the graywacke sample due to flooding was about one per cent of the initial sample height and that due to jetting was about 0.4 per cent of the initial height. These amounts of settlement should not be considered quantitatively as it is difficult to obtain uniformity between samples when placing the rock in the consolidometer. Although all samples of a particular rock type had the same gradation and approximately the same initial void ratio, slight variations in position of the particles

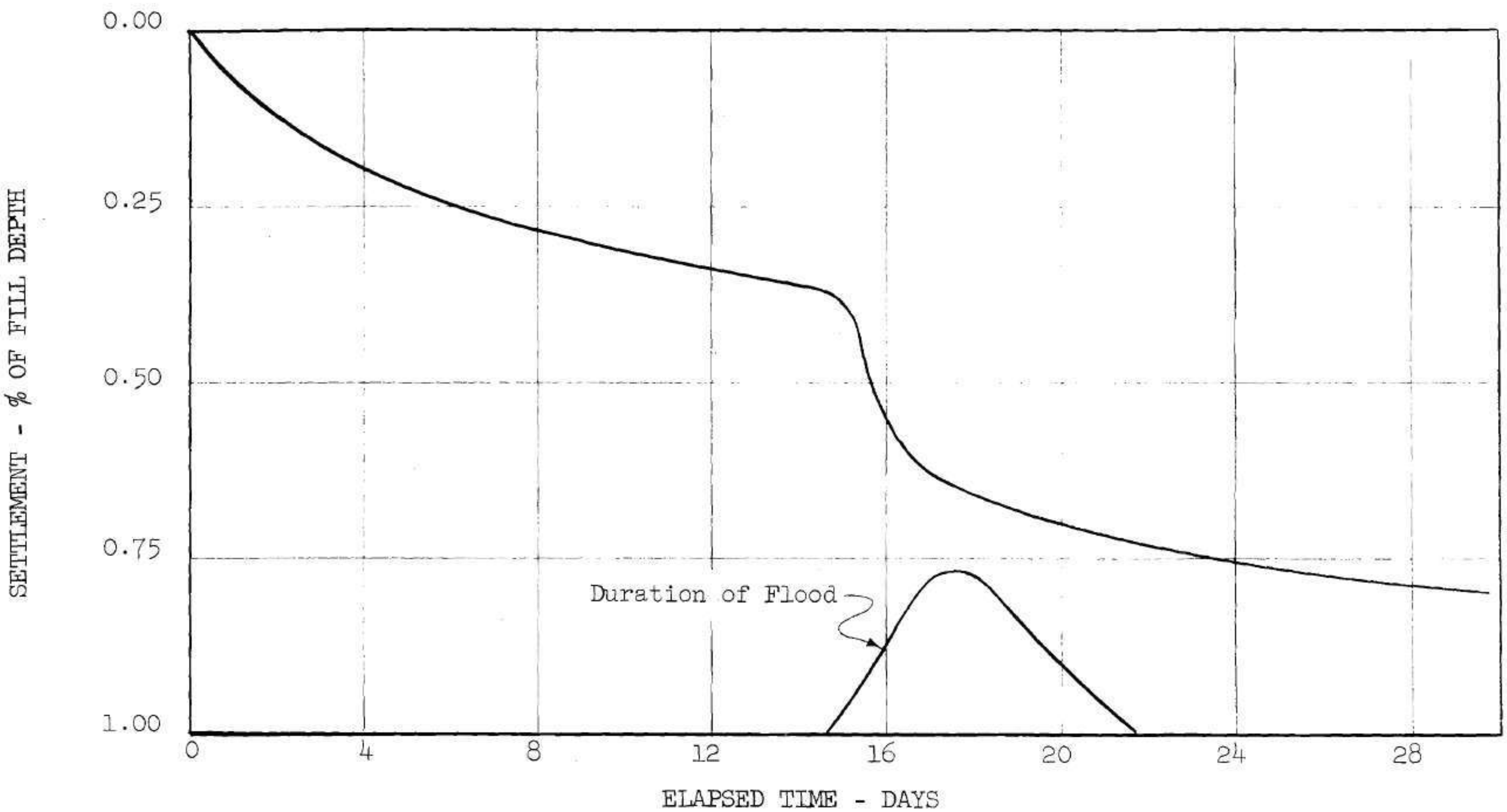

Figure 10 Time - Settlement Curve for Point on Dix River Dam

in the consolidometer caused considerable differences in settlement characteristics. While most tests yielded reasonably close results, the flooding test mentioned (Test 2) settled far more than any of the other similar tests, including those using Pottsville Sandstone--a far weaker material than the Nantahala Graywacke (see Figure 9). This single result should not lead to the conclusion that flooding is more beneficial to increased settlement than jetting.

The flooding of samples after application of load effectively reproduces the sequence of events that takes place with a rockfill dam constructed without sluicing. Dix River Dam in Central Kentucky was ineffectually sluiced during construction (2), p. 10, and when construction reached the 185 foot level and the dam was subjected to a flood that flowed freely through the lower 60 feet of the fill, the effect was practically immediate. Figure 10 shows the resulting effect on a point maintained at the top of the fill (17), p. 44. The similarity between this curve and the curves of Tests 2, 5 and 6 is evident.

The jetting of samples after application of load has no practical significance but was performed merely to see if the jetting action, by moving the smaller fines and dust created during point fracture, had any additional benefit over the flooding process. The indications are that the greatest value of jetting was in the wetting of the particles, which was more effectively accomplished by flooding.

Test 5, as plotted in Figure 11, was continued by flooding the sample after the settling effect from jetting was negligible. The results indicate that the second application of water had little if any effect toward further settlement. This result is supported by Test 6

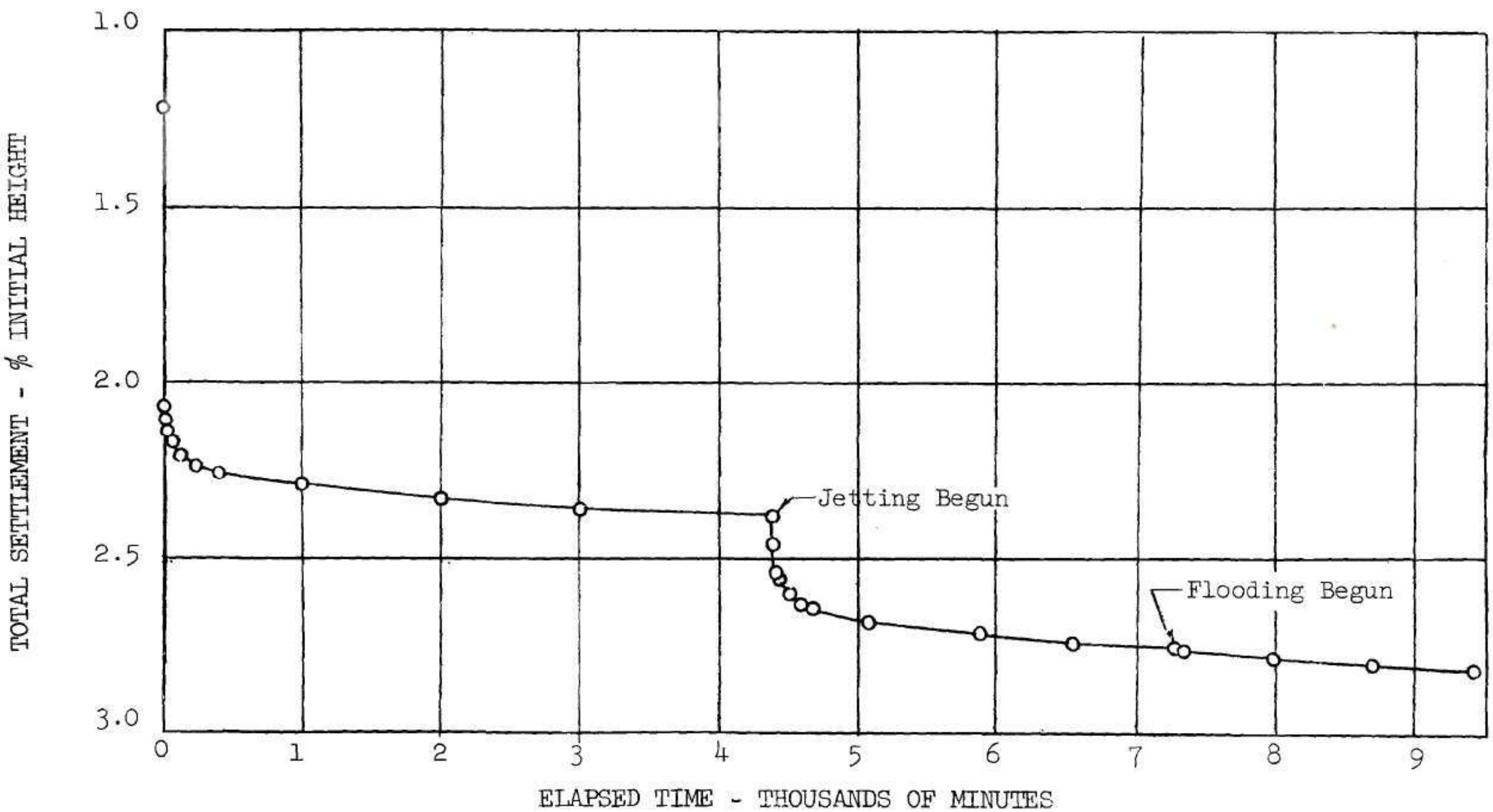

Figure 11. Time - Settlement Curve For 16,298 psf Loading - Nantahala Graywacke
Test No. 5

(see Figure 9) which was run on pre-saturated sandstone. Jetting with water at about 4400 minutes during the 5000 pound loading produced no additional settlement.

Effect of Rock Type

The results of this comparison between the settlement characteristics of Nantahala Graywacke and Pottsville Sandstone are not conclusive due to the large amount of settlement experienced by Sample 5. It would seem that the manner in which the rock comes to rest in the mass is of more importance than the strength of the rock (at least for the rocks tested). The other tests (Figure 9) yielded the expected results that the immediate settlement of the sandstone was considerable greater than that of graywacke. It is interesting to note that the amounts of secondary settlement undergone in a given period of time by the two rocks are quite similar. Between 1000 and 4000 minutes, Sample No. 2 (graywacke) and Sample No. 6 (sandstone) each settled 0.09 per cent of their respective heights. This is a logical result because once the immediate settlement is complete, sufficient fracture has occurred in point and edge contacts--regardless of material--for the load to be distributed throughout the mass without further extensive fracturing. From this point on, the settlement of each mass becomes dependent on the tendency of the rock in the individual contacts to fracture with time due to shifting loads, vibration and/or creep, with the subsequent rotation and translation of particles due to this fracture and the forming of new point and edge contacts. Therefore, the strength of the rock is not the major factor in the secondary settlement rate of rockfill. This is evident in the records of existing rockfill dams built of rock of widely varying

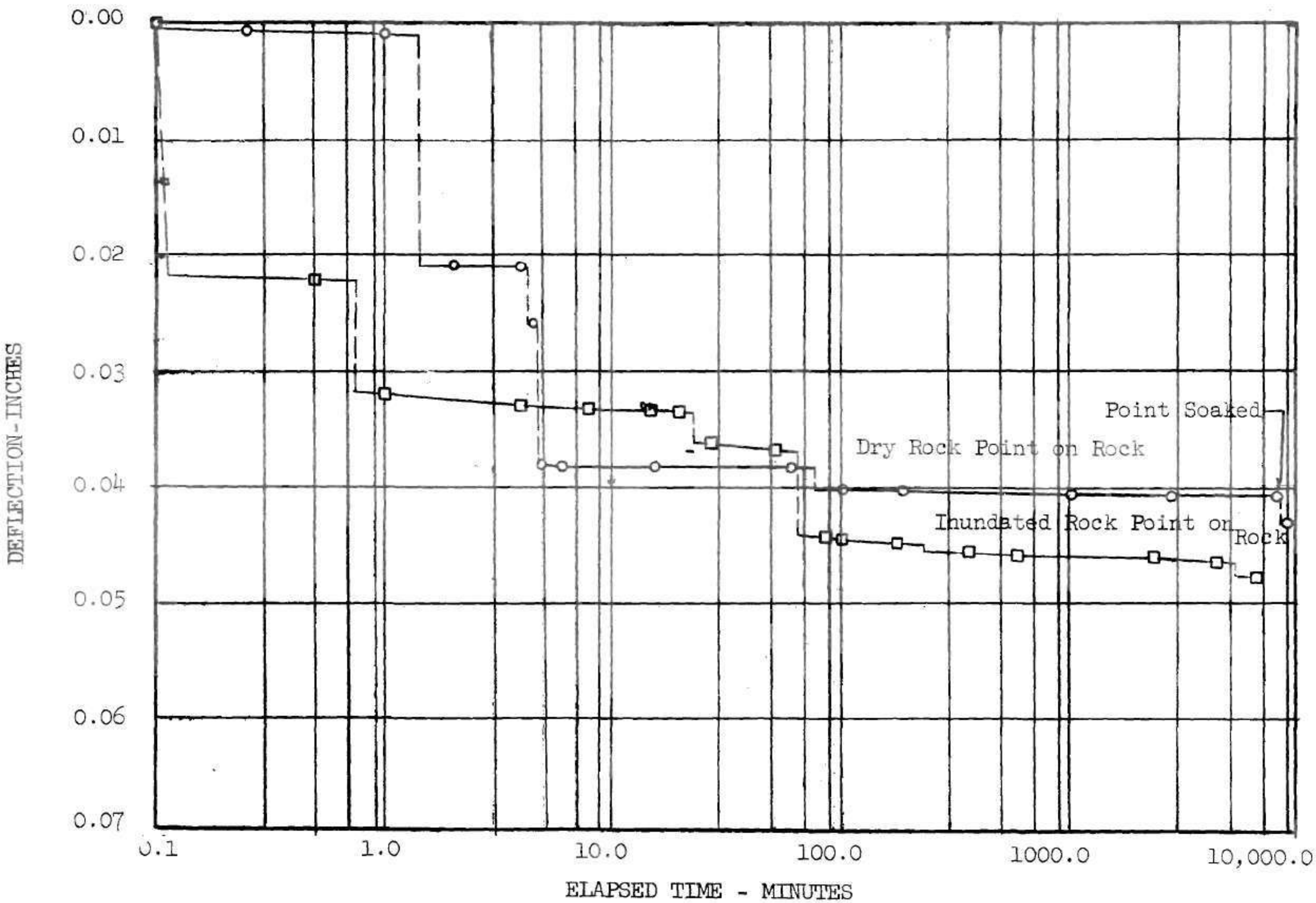

Figure 12a. Point-Crushing Tests - Nantahala Graywacke Points Under 100 lb Load

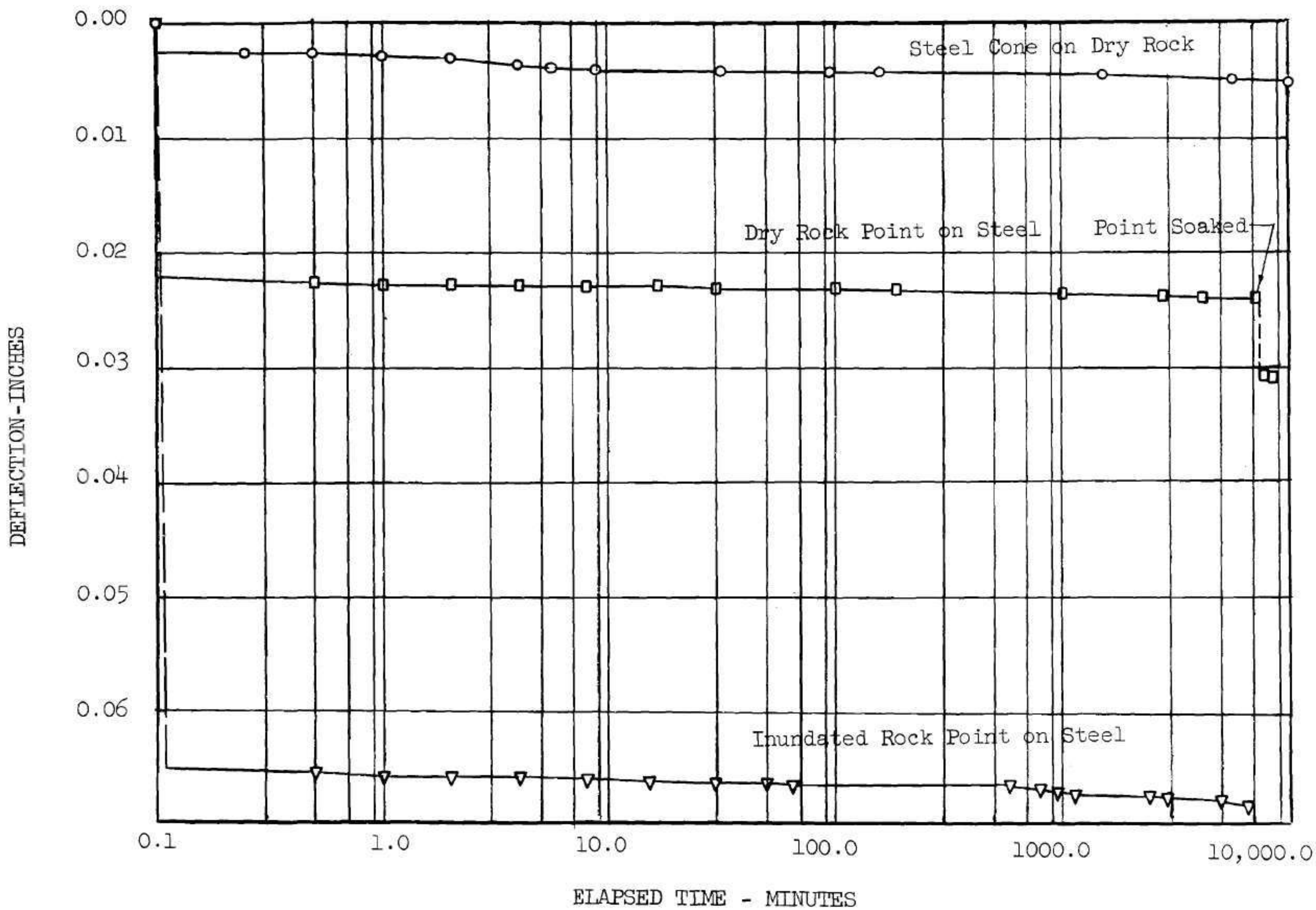

Figure 12b. Point-Crushing Tests - Steel and Nantahala Graywacke Points Under 100 lb Load

strengths, but all having settlements generally within one per cent of each other. The settlement records of dams generally do not show immediate settlement.

This concept of secondary settlement of rockfill will be discussed more fully later.

Effect of Pre-Saturation

A comparison of Tests 6 and 7 (see Figure 9) shows that the immediate settlement of the pre-saturated sandstone was slightly greater than that of the dry sample. The rate of settlement of the pre-saturated sample was greater than that of the dry one for the first part of the test, but after about 2000 minutes the pre-saturated sample attained a slower rate of secondary settlement than the dry sample, which maintained a constant rate throughout the secondary settlement part of the curve.

Point-Crushing Tests

It would appear that a rock point bearing against a hard surface and subjected to a constant load of sufficient magnitude would immediately fracture until the bearing area of the point was sufficiently great to transmit the load, after which no further action could be expected. The results of this series of tests indicate that this is not the case.

It was found that the graywacke points bearing on graywacke slabs fractured as many as five times under constant loads of 100 pounds during the tests, which ran for about one week. The dashed vertical lines in Figures 12a and 12b indicate the sudden failure of the rock. They were plotted as dashed lines because the exact time of failure was unknown having been sometime between readings.

The inundated rock point bearing on rock showed greater total deflection than the dry rock bearing on rock. This was to be expected, as the weakening effect of water on rock is well known. Rock points bearing against steel were tested to see if any of the deflection in the rock on rock tests was due to local failure in the rock slab. The results indicate that this was happening, at least in the dry rock on rock test. This was also substantiated by a post-test examination of the rock slab under a binocular microscope. Figures 13 and 14 show respectively a microscopic view of a graywacke point after testing against a graywacke slab, and a microscopic view of that slab, showing the debris surrounding the contact point.

It is interesting to note that the points bearing against steel fractured only once, the inundated point against steel deflecting considerably more than did the inundated rock point on rock. This test is a good example of why caps are needed in the consolidation tests to prevent steel plates from bearing directly against the rock.

A possible explanation for the multiple fracturing in the rock on rock tests might be that these tests involve failure of both the rock in the point and the rock in the slab. It appears from the steel cone on dry rock test that failure of the rock in the slab was not a sudden fracture of any magnitude. However, even a slight change in the bearing surface could change the loading of the point sufficiently to bring about failure. Similarly, the fracture of the point could cause the load on the slab to be shifted slightly, resulting in newly stressed rock in the slab and a continuation of the process. This exchange of failure did not have a chance to develop with the rock on steel tests as the fracture of

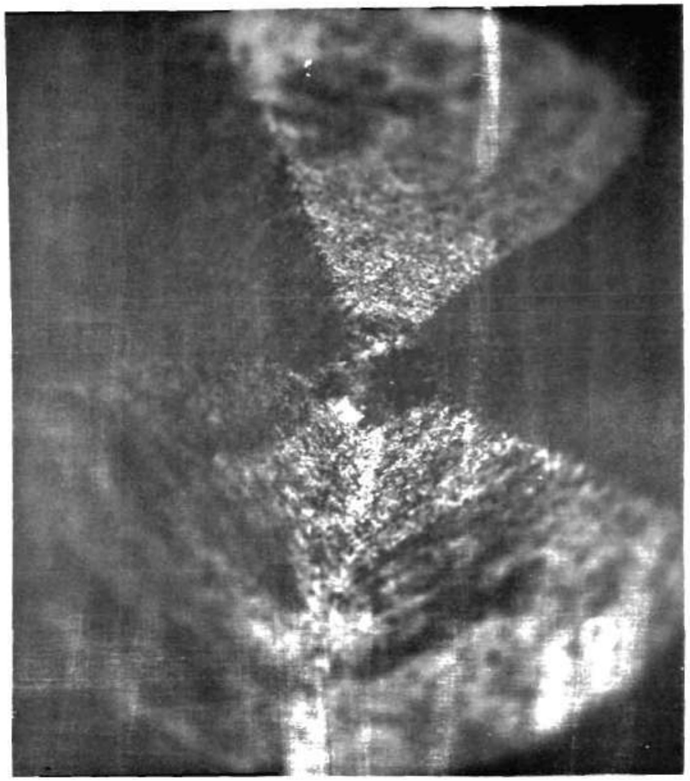

Figure 13. View of Graywacke Point After Testing (x50)

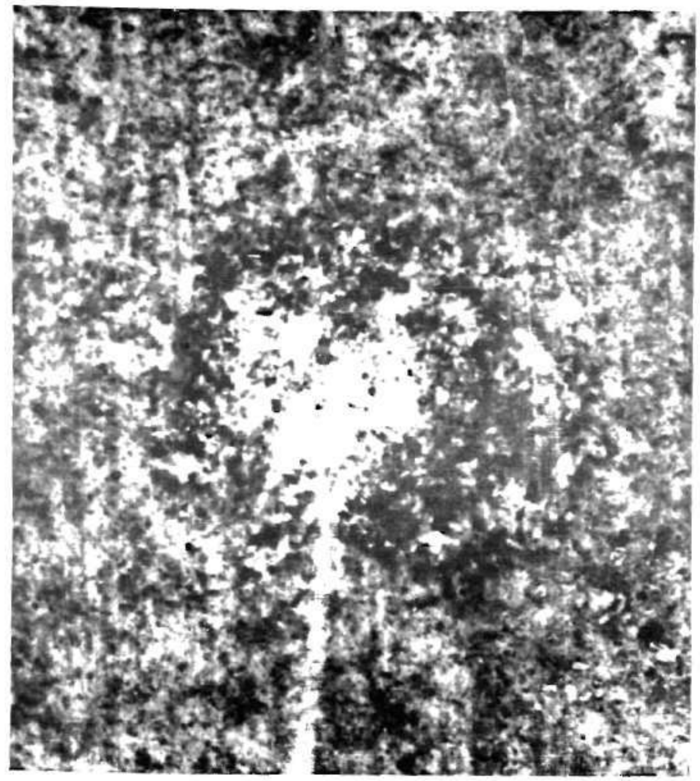

Figure 14. View of Contact Point on Graywacke Slab After Testing (50x)

the rock point had little effect on the steel plate.

It should not be assumed that just because the rock on steel tests did not indicate multiple fractures during the testing period, they would not have taken place with time. Similarly, it should not be assumed that the rock on rock tests could not experience additional fractures if allowed to continue. Existing rockfill dams contain point and edge contacts that have been continuously stressed for over 50 years, and settlement records suggest that this multiple fracturing is continuing.

In all of these six tests there was gradual deflection that continued throughout the tests--including the time between abrupt fractures in the rock on rock tests. There are two possible explanations for this. Either the rock is undergoing numerous small failures that appear as a continuous deflection with time, or the rock is undergoing creep, which will be defined as continuous strain while maintaining load, when subjected to a continuing stress.

Creep Tests

Temperature variations prevented any valid determination of creep using the micrometer dial gage. The attempt to measure creep in graywacke with SR-4 strain gages affixed to a cylinder of the rock also met with failure. The unconfined compression tests on the graywacke cylinders yielded widely varying values for ultimate strength (see appendix, Figure 16). This presented a problem when an attempt was made to subject the cylinder with its gages to a high stress without fracturing it, when the actual ultimate strength of that particular cylinder was unknown. It was stressed to 23,000 psi, which it maintained for approximately five minutes. Unfortunately, while strain readings were being made, the cylinder fractured. Time limitations did not permit another attempt to measure creep.

This failure, however, is significant in itself. The graywacke cylinder did maintain the load for about five minutes before breaking. A similar graywacke cylinder, maintained under a constant compressive stress of 31,667 psi, failed between three and four hours after loading, and another cylinder which was stressed constantly to 28,367 psi fractured after seven hours and three minutes. These time-dependent failures emphasized the need for a closer examination of the graywacke to determine the mechanism of failure.

Microscopic Analyses

The examination of the thin sections under the polarizing microscope revealed several interesting points. The grains in the graywacke were generally irregular in shape and varied in size from about 540 microns to 10 microns in greatest dimension, with an average size of about 190 microns. The grains were tightly interlocked with very little if any obvious cementing. There was no evidence of fusion or crystalline bonding, thus the strength of the rock appears to be due mainly to contact adherence between the grains. All three sections indicated a great percentage of quartz, some feldspar and varying amounts of muscovite and biotite mica, among other minerals, which caused the differences in visual appearance of the three specimens. The darker the specimen, the more biotite mica it contained.

It was immediately evident why the writer was unable to saturate the graywacke. The porosity of the rock appeared to be a fractional part of one per cent, and could be considered negligible (see Figure 15).

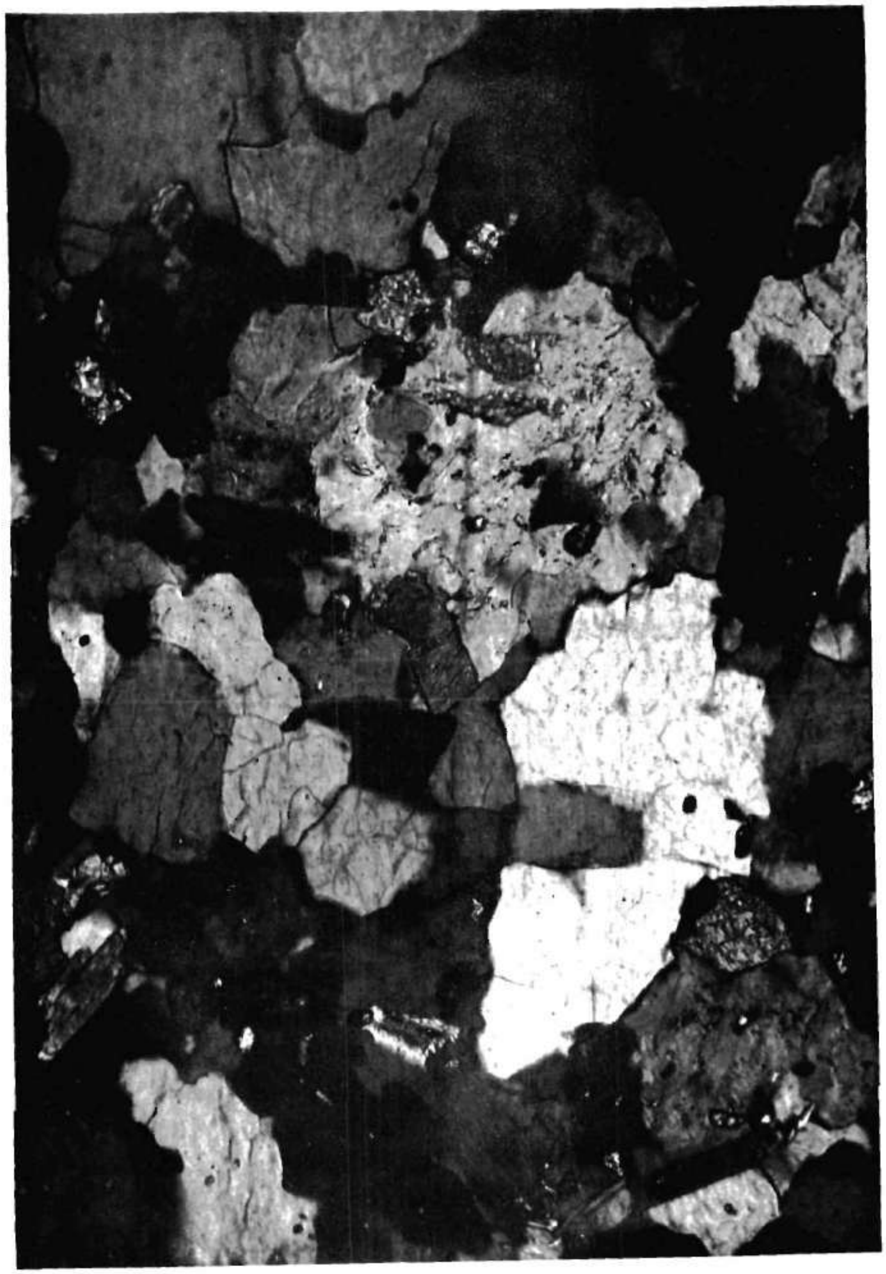

Figure 15. Photograph of a Thin Section of Nantahala Graywacke Through a Polarizing Microscope (50x)

No detailed petrographic analysis of the sections was attemped, as it was not considered to be a necessary part of this research.

A microscopic examination of the shear surfaces of the fractured cylinders, as compared to unfractured surfaces of the same material, revealed an apparent orientation of the mica flakes parallel to shear surfaces, while the mica visible in the unfractured surfaces appeared to be more randomly oriented.

The large percentage of quartz in the graywacke and the small amount of cementing agent, as revealed by this cursory examination, indicate that perhaps this material should be defined as a subgraywacke, however, a more quantitative analysis would be required before this decision could be made.

Mode of Failure

If a critical strain theory of failure is accepted as being applicable to graywacke tested in unconfined compression, the reason for the time-dependent failure of the three cylinders is apparent. In each test the highly stressed rock cylinders crept until a certain critical strain was reached, at which point the cylinders broke.

It appears that any creep occurring in the graywacke is due mainly to slippage between the grains--particularly in the regions where smaller round particles are in contact with larger particles, thus affording minimum contact adherence between grains. The rupture of any intergranular cementing agent present is undoubtably involved, but this appears to be a minor factor. The presence of oriented mica, however, which was evident in every shear surface examined, suggests that the mica flakes slowly bend, as the grains slip under constant load, until planes of weakness are established causing the graywacke to fracture.

Thus the critical strain mentioned above as being necessary for failure might actually be the movement between grains necessary to produce orientation of the mica flakes into planes of weakness. This would help to explain why such widely varying results were obtained for ultimate strength of the cylinders in unconfined compression. Although the cylinders were cored from the same piece of rock, it was noted that there were variations in the mica content from cylinder to cylinder. The cylinders with less mica would not form planes of weakness as quickly as would those containing more mica.

The Effect of Water

The unconfined compression tests of the graywacke cylinders indicated a reduction in average strength for the wet cylinders (see Table 1); however, the scatter of results for the wet tests was in the same range as the dry test results. Since the cylinders could not be saturated, any reduction in strength must have been due to weakening of the outermost layer of rock (which was observed to chip off), thus reducing the cross-sectional area of the cylinders and increasing the stress.

No reduction in strength was observed due to saturation of the sandstone cylinders (see Figure 16). This result was substantiated by similar test results of Schwartz (33), p. 41.

Near the end of the point-crushing tests, both of the dry rock points were wet with water applied by means of a squeeze bottle. Within a few minutes they each broke again (see Figures 12a and 12b), substantiating the theory that water weakens rock.

An explanation for these results is not apparent to the writer.

It has been shown by the futile attempts to saturate the graywacke and by the thin section examination that there are no interconnecting voids in that material and, in fact, are very few voids at all. In addition, it was discovered that there is very little cementing agent between the grains. Thus it is difficult to understand how Terzaghi's conditions for weakening of rock by water could apply to this material. If there is very little intercrystalline matrix, then the application of water could not weaken to any great degree the intercrystalline connections by dissolving the soluble part of the matrix. Also, if there are very few if any interstices in the rock and certainly no interconnected interstices, then the strength of the rock is not due to surface tension in the interstitial liquid, and immersion could not eliminate this strength.

The increased settlement of rockfill due to application of water--which was observed repeatedly in the consolidation tests--must be attributed at least in part to weakening of the rock, as this was shown to take place in the point-crushing tests. It is known (23) that water is not a lubricant, however, the possibility of clay particles, present in the rock due to weathering, dissolving to form a "soapy" surface should be considered.

In conclusion, it should be pointed out that, while some point and edge contacts in a rockfill interior will overbreak under pressure and form contact areas greater than necessary to support the load--thus being at stresses considerably less than critical--others will undoubtedly fracture just enough to support the load at very close to the critical stress. They will, in effect, be in a state of "precarious equilibrium" in which the slightest increase in pressure, any shock or vibration, or a slight weakening of the rock will cause them to fail. Thus the application of water to the mass might have enough effect to cause failure of some points or edges. This effect might be weakening of the rock in the contacts as mentioned above, or possibly just a shift in the direction of load applicati on due to the bouyant effect of the water. The failure of one point may change the equilibrium conditions of that particle and surrounding particles, the end result being more settlement.

CHAPTER V

CONCLUSIONS

The testing program undertaken was too broad in scope for the available time to allow the writer to draw definite conclusions in some areas, based upon the results of a limited number of tests.

(1) There are indications that an increase in uniformity and size of rock particles in a rockfill results in greater immediate settlement and a smaller rate of secondary settlement.

(2) The application of water to a rockfill by any means, after it is constructed dry, will bring about an increase in settlement. The increased value of high-pressure sluicing over flooding in moving fines beyond the top layer of rock is seriously questioned.

(3) There are indications that the rate of secondary settlement of a rockfill may be independent of the strength of the rock.

(4) Pre-saturation of the rock particles was shown to have about the same effect on settlement as the application of water by sluicing or flooding.

(5) Points of Nantahala Graywacke bearing on rock surfaces and subjected to constant, sustained loads of sufficient magnitude to cause fracture, will fracture more than once with time, and, between fractures, will exhibit a slow deflection that may be creep.

(6) There are indications that Nantahala Graywacke cylinders, under constant, sustained high loadings, will fail with time. It is hypothesized that the failure may be dependent on some critical strain

that varies with the mica content of the graywacke, and the orientation of that mica.

(7) Water has a weakening effect on Nantahala Graywacke, but the mechanics of this process are not evident to the writer.

(8) The study of settlement of rockfill in the laboratory using small models of crushed rock necessitates the running of many similar tests so that statistical results may be obtained. This is required because of the impossibility of constructing identical samples of crushed rock.

CHAPTER VI

RECOMMENDATIONS FOR FURTHER STUDY

(1) Conclusion 7 above leads to the general recommendation that more consolidation tests be undertaken to clarify the effects of gradation and of rock strength on the settlement characteristics of a rockfill.

(2) Further efforts should be made to measure creep in Nantahala Graywacke.

(3) An investigation into the cause of rock weakening by water should be made.

A P P E N D I X

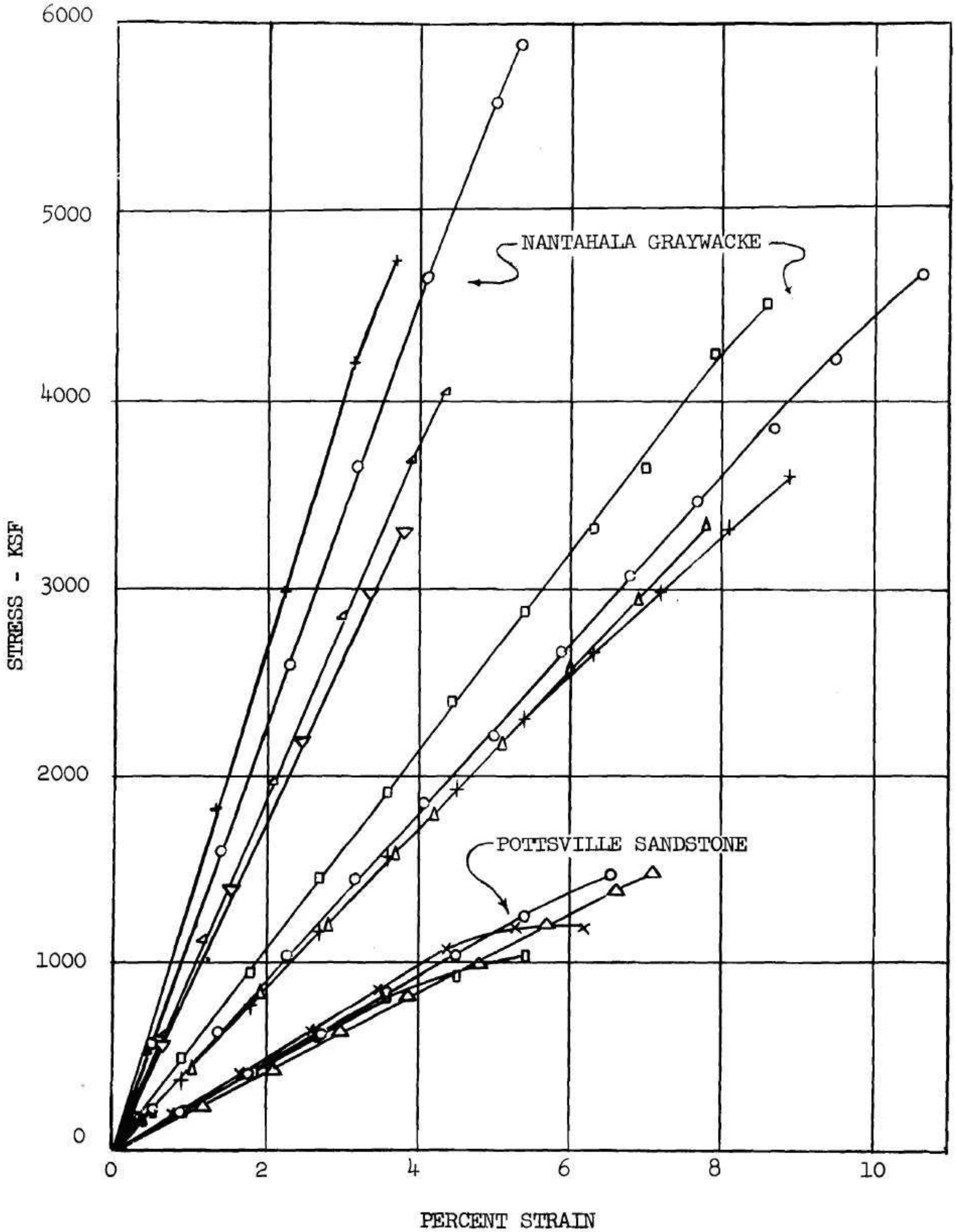

Figure 16. Stress - Strain Curves for Rocks Tested in Unconfined Compression

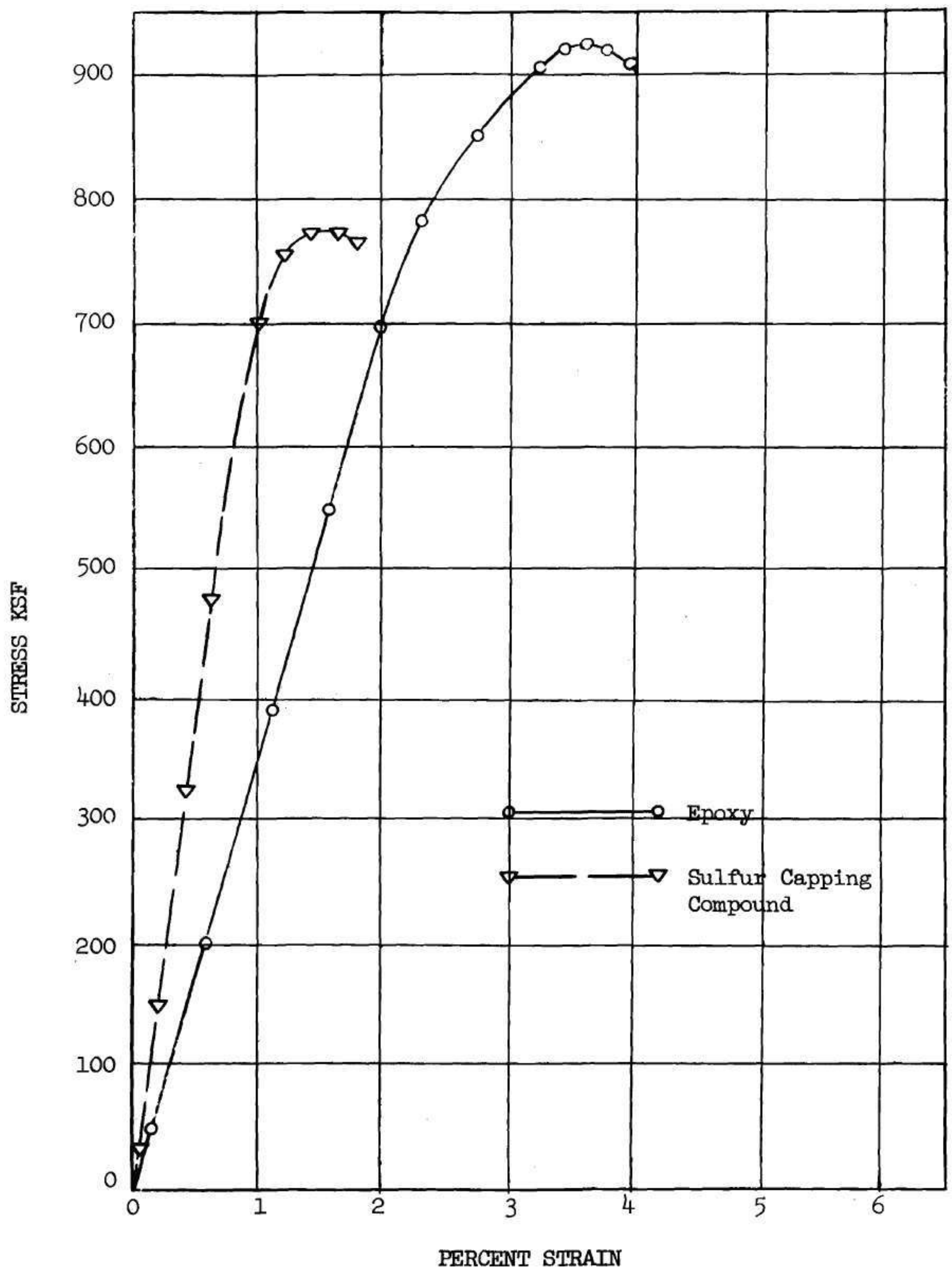

Figure 17. Stress - Strain Curves for Capping Materials in Unconfined Compression

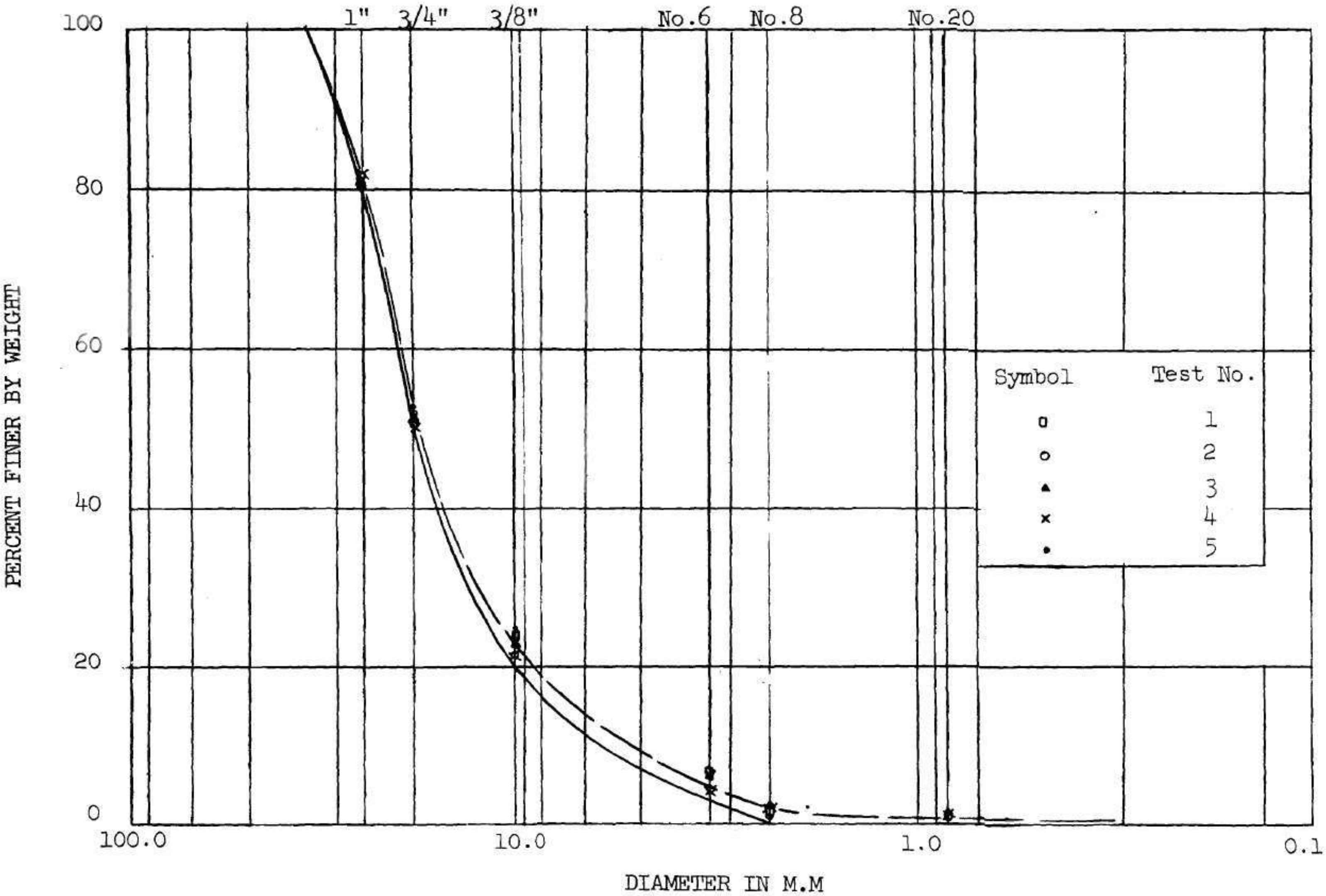

Figure 18. Particle Size Distribution Curves - Nantahala Graywacke Consolidation Tests

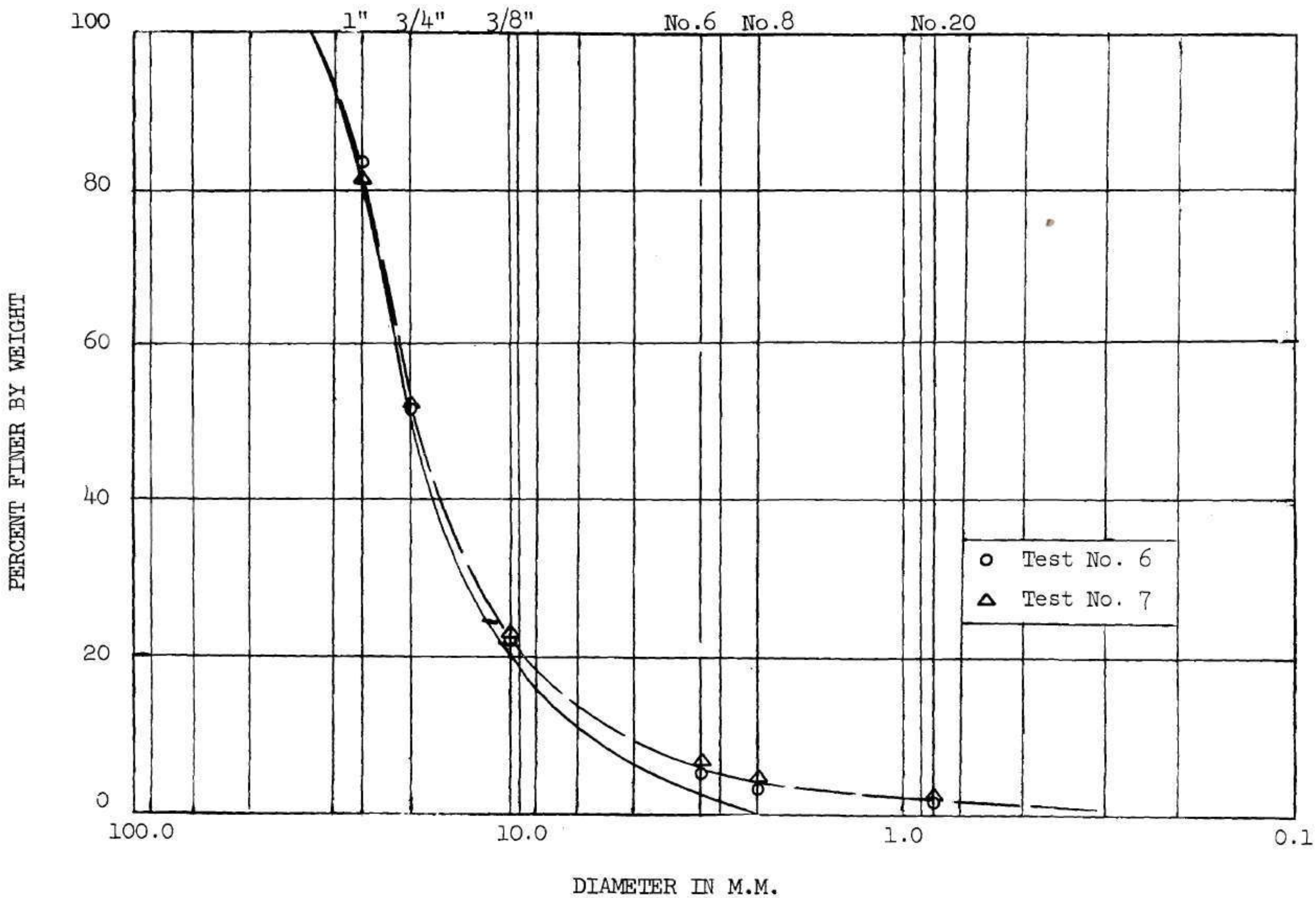

Figure 19. Particle Size Distribution Curves - Pottsville Sandstone Consolidation Tests

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